ERDC/CHL TR-02-18

Coastal and Hydraulics Laboratory

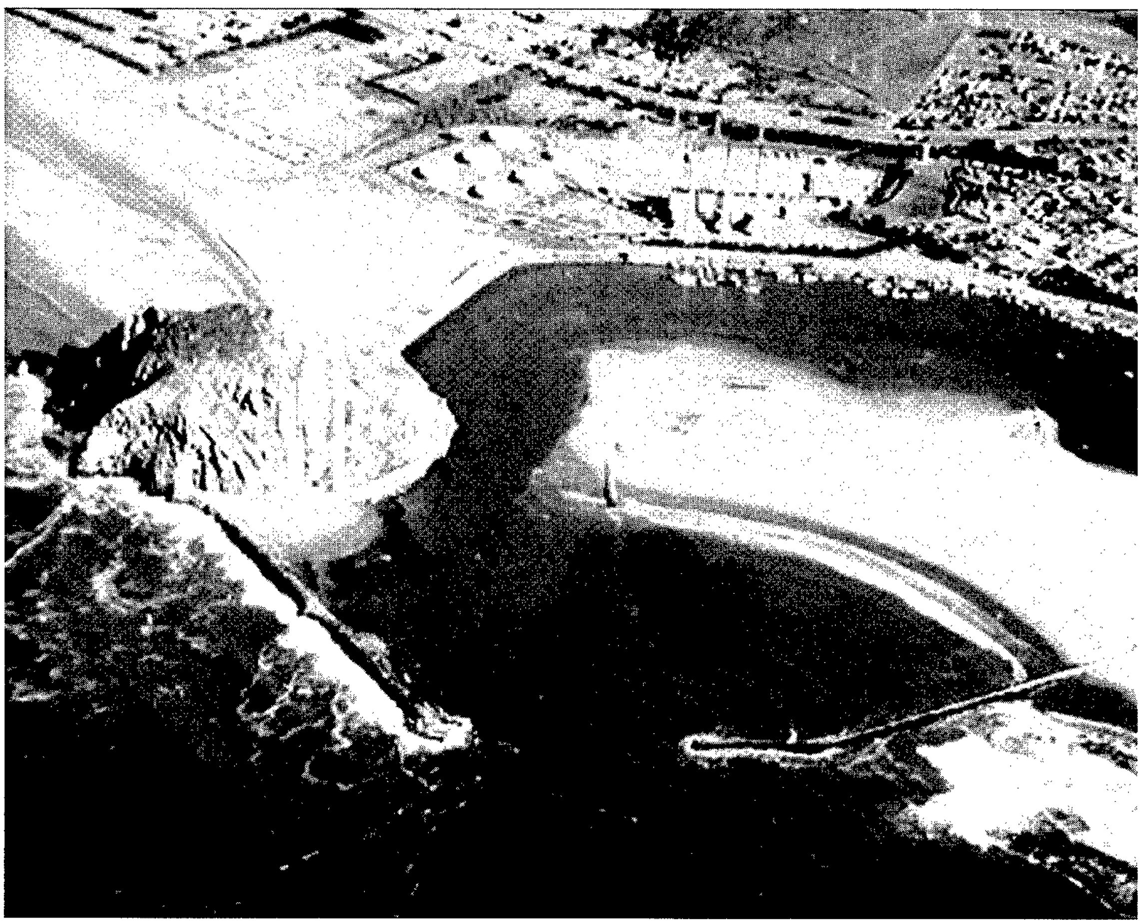
US Army Corps of Engineers®
Engineer Research and Development Center

Monitoring Completed Navigation Projects Program

Monitoring of Entrance Channel Navigation Improvements at Morro Bay Harbor, Morro Bay, California

Edward F. Thompson, Robert R. Bottin, Jr., and Arthur T. Shak

September 2002



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Monitoring of Entrance Channel Navigation Improvements at Morro Bay Harbor, Morro Bay, California

by Edward F. Thompson, Robert R. Bottin, Jr.

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Preface

The study reported herein was conducted as part of the Monitoring Completed Navigation Projects (MCNP) Program (formerly Monitoring Completed Coastal Projects) Program. Work was carried out under Work Unit 11M14, "Morro Bay Monitoring Study, California." Overall program management for MCNP is accomplished by Headquarters, U.S. Army Corps of Engineers (HQUSACE). The Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), is responsible for technical and data management and support for HQUSACE review and technology transfer. Program Monitors for the MCNP Program are Messrs. Barry W. Holliday, Charles B. Chesnutt, and David B. Wingerd (HQUSACE). The Program Manager is Mr. Robert R. Bottin, Jr. (CHL).

The work was conducted during the period January 1998 through August 2001 under the general supervision of Dr. James R. Houston, former Director, CHL, and Mr. Thomas W. Richardson, Director, CHL, and under direct supervision of Mr. Dennis G. Markle, Chief, Coastal Harbors and Structures Branch, CHL. Principal Investigators for the study were Mr. Bottin, research physical scientist, CHL, Dr. Edward F. Thompson, research hydraulic engineer, CHL, and Mr. Arthur T. Shak, supervisory hydraulic engineer, U.S. Army Engineer District, Los Angeles (CESPL). This report was prepared by Dr. Thompson and Messrs. Bottin and Shak.

Limited ground surveys, aerial photography, and photogrammetric analysis of the south breakwater were conducted by Richard B. Davis, Inc., Smith River, CA, and bathymetric surveys of the harbor were conducted by Los Angeles District personnel. Additional aerial photography, multibeam and side-scan sonar were completed for the north and south breakwaters by Racal/Pelaegos, Inc., Irvine, CA. Dredging was completed by NATCO Limited Partnership of Oak Brook, IL, and Manson Construction Company of Seattle, WA, along with the Government dredge *Yaquina.*

Acknowledgements also are extended to the following for their contributions during the study as noted:

Development and deployment of prototype gauges: Dr. Andrew W. Garcia, Messrs. Michael W. Tubman, Larry G. Caviness, and Ralph E. Ankeny, CHL

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Numerical modeling work: Messrs. Bratos and S. Jarrell Smith and Drs. Thompson, Zeki Demirbilek, and Joseph Z. Gailani, CHL

Director of ERDC during the conclusion of this investigation and the publication of this report was Dr. James R. Houston. COL John W. Morris III, EN, was Commander and Executive Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in figures, plates, and tables of this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet	28.32	liters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	30.48	centimeters
feet	0.3048	meters
inches	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square meters
square miles (U.S. statute)	2.589988	square kilometers
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

Monitoring Completed Navigation Projects Program

The goal of the Monitoring Completed Navigation Projects (MCNP) Program (formerly Monitoring Completed Coastal Projects Program) is the advancement of coastal and hydraulic engineering technology. It is designed to determine how well projects are accomplishing their purposes and are resisting attacks of the physical environment. These determinations, combined with concepts and understanding already available, will lead to more credibility in predicting engineering solutions to coastal and hydraulic problems; to strengthening and improving design criteria and methodology; to improving construction practices and cost-effectiveness; and to improving operation and maintenance techniques. Additionally, the monitoring program will identify where current technology is inadequate or where additional research is required.

To develop the direction for the program, the U.S. Army Corps of Engineers established an ad hoc committee of coastal and hydraulic engineers and scientists. The committee formulated the program's objectives, developed its operational philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed, essentially a listing of the program's areas of interest. Subsequently, an engineer regulation (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1997) was developed that governs the program.

Corps Division offices are invited to nominate projects for inclusion in the monitoring program as funds become available. A selection committee reviews and prioritizes the projects nominated based on criteria covered in the engineer regulation. Projects are then reviewed by members of the MCNP Program Field Review Group (representatives from District and Division offices). The prioritized list finally is reviewed by the Program Monitors at HQUSACE. Final selection of projects to be monitored is based on this prioritized list, national priorities, and the availability of funding.

The overall monitoring program is under the management of the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), with guidance from HQUSACE. Operation of individual monitoring projects is a cooperative effort between the submitting

District/Division office and CHL. Development of monitoring plans and the conduct of data collection and analyses are dependent upon the combined resources of CHL and the District/Divisions. Morro Bay Harbor, CA, was nominated and subsequently approved for inclusion in the MCNP Program in 1997.

Project Location and History

Morro Bay Harbor is located in a natural embayment on the central coast of California about midway between Los Angeles and San Francisco (Figure 1). It serves as the only all-weather small craft commercial/recreational harbor between Santa Barbara and Monterey. Morro Bay extends inland and parallels the shore for a distance of about 6.4 km (4 miles)[1] south of its entrance at Morro Rock. The bay is approximately 1.6 km (1 mile) wide and has an area of about 9.1 sq km (3.5 square miles). A sandspit, about 6.4 km long (4 miles long) by 0.8 km wide (0.5 miles wide), separates Morro Bay from the ocean. The harbor is protected from the effects of the open ocean by a Federal navigation project consisting of two permeable, rubble-mound breakwaters, an inner harbor groin, and a stone revetment. The Morro Bay Federal Project also has 4,175 m (13,700 ft) of navigation channels, as shown in Figure 2.

The north breakwater is 575 m long (1,885 ft long) with an average crest elevation (el)[2] of +5.5 m (+18 ft), while the south breakwater is 558 m (1,832 ft) in length with a crest el varying from +4.3 to +5.5 m (+14 to +18 ft). These breakwaters are positioned to form a 274-m-wide (900-ft-wide entrance). Other structural features include a 489-m-long (1,600-ft-long) stone revetment located adjacent to Morro Rock, a 305-m-long (1,000-ft-long) stone groin located along the north end of the sandspit adjacent to the entrance channel, and a stone revetment extending northeasterly adjacent to Navy Channel. The Federal navigation channel commences at the gap formed by the outer breakwaters and extends to the lower bay via three channel reaches. Prior to the latest improvements, the authorized entrance channel depth was -4.9 m (-16 ft) and the innermost channel was maintained at a depth of -3.7 m (-12 ft).

Prior to the latest entrance channel improvements, Morro Bay Harbor was known as one of the most dangerous harbors in the United States. Since 1962, 20 deaths, 67 injuries, and more than $600,000 in vessel damages had resulted from accidents caused by steep and breaking wave conditions in the harbor entrance. The harbor experienced entrance problems due to a combination of exposure to storm wave conditions from the Pacific Ocean and bathymetry in the vicinity of the entrance. Breaking waves occurred at the entrance when incident wave heights exceeded 3 m (10 ft). Hazardous conditions also were reported for 2.4 to 3-m (8 to 10-ft) waves, which tended to steepen sharply when they reached

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[1] All units of measurement are in SI units followed by non-SI units in parenthesis. A conversion table for converting non-SI to SI units used in figures, tables, and plates is on page ix.

[2] All elevations (el) and depths cited herein are referenced to mean lower low water (mllw) datum.

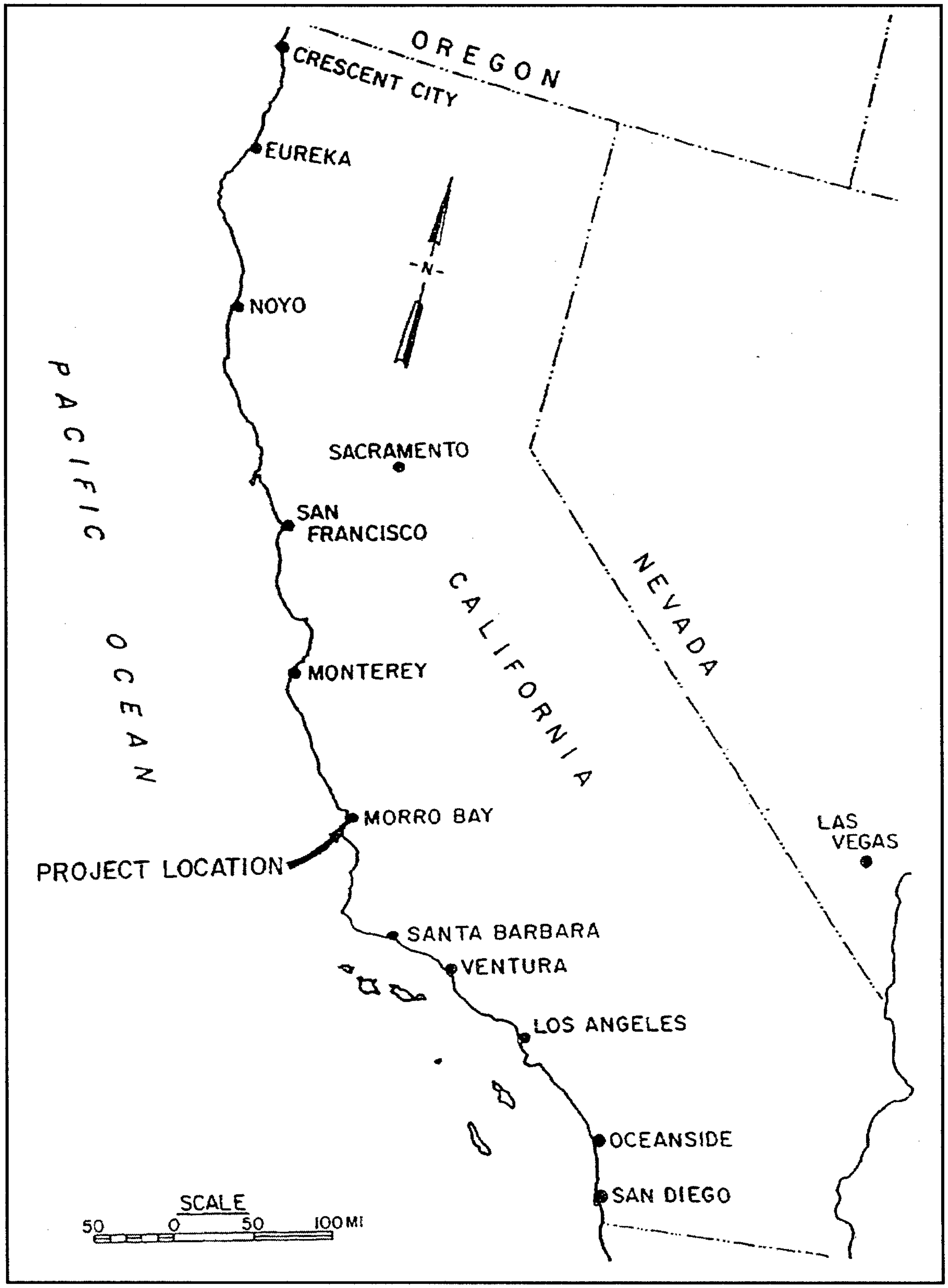

Figure 1. Project location

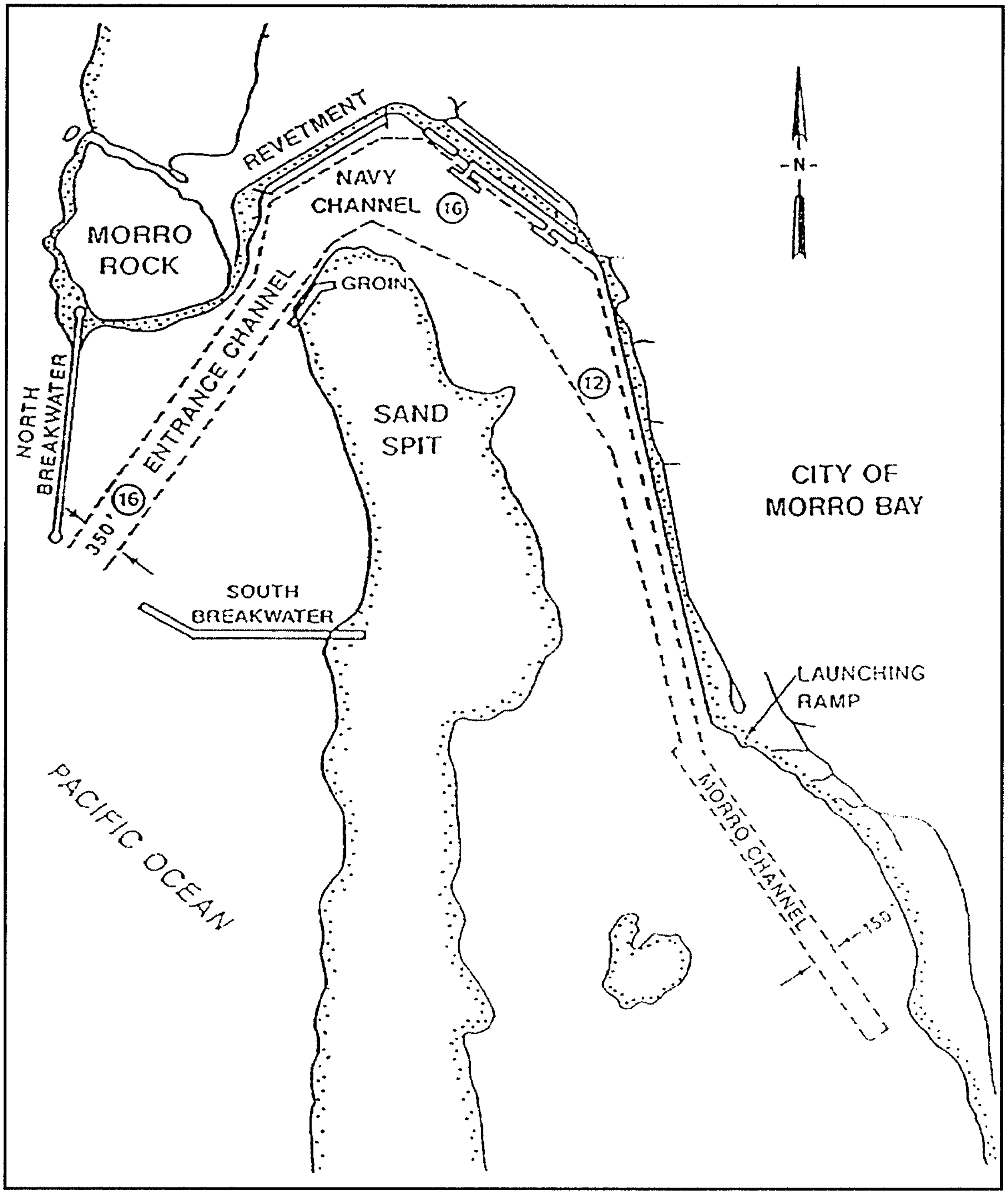

Figure 2. Morro Bay Federal project

the shallower harbor entrance, particularly during ebb tide conditions. An aerial view of the harbor entrance is shown in Figure 3.

A feasibility study was completed by the U.S. Army Engineer District, Los Angeles (1991) to provide safer navigation by mitigating undesirable steep and breaking wave conditions in the entrance. A wide array of navigation improvements was investigated, including breakwater extensions, detached

Figure 3. Aerial view of Morro Bay entrance

offshore breakwaters, and modifications to the Federal entrance channel. The breakwater alternatives lacked economic justification and were eliminated from consideration. The channel modifications were viable and economically justified. Channel modification plans were expected to allow most large waves to pass through the entrance to Morro Bay Harbor without breaking, or steepening, and creating hazardous conditions. The final design is given by USAED, Los Angeles (1994).

The latest improvements at Morro Bay entrance were completed in December 1995 and consisted of construction of a deepened, expanded entrance channel. The new channel doglegs westerly from the old entrance channel and flares open to a width of 290 m (950 ft). The authorized depth of the channel extension is -9.1 m (-30 ft). However, the plan provides for advanced maintenance by deepening the new channel to -12.2 m (-40 ft) and dredging an additional sand trap to a depth of -9.1 m (-30 ft) within the harbor entrance structures north of the head of the south breakwater. A plan view of the 1995 entrance channel improvements is shown in Figure 4.

Predicted Design Performance

Numerical and physical model investigations were conducted at ERDC to optimize project design performance at the harbor entrance. Numerical studies were conducted during the period January - May 1989 (Kaihatu, Lillycrop, and Thompson 1989) and a physical model investigation was conducted from June - September 1992 (Bottin 1993). The Los Angeles District also conducted numerical investigations during their feasibility study to predict design performance. In addition, the Los Angeles District conducted a limited field investigation involving dredging of a test trench to estimate longshore sediment

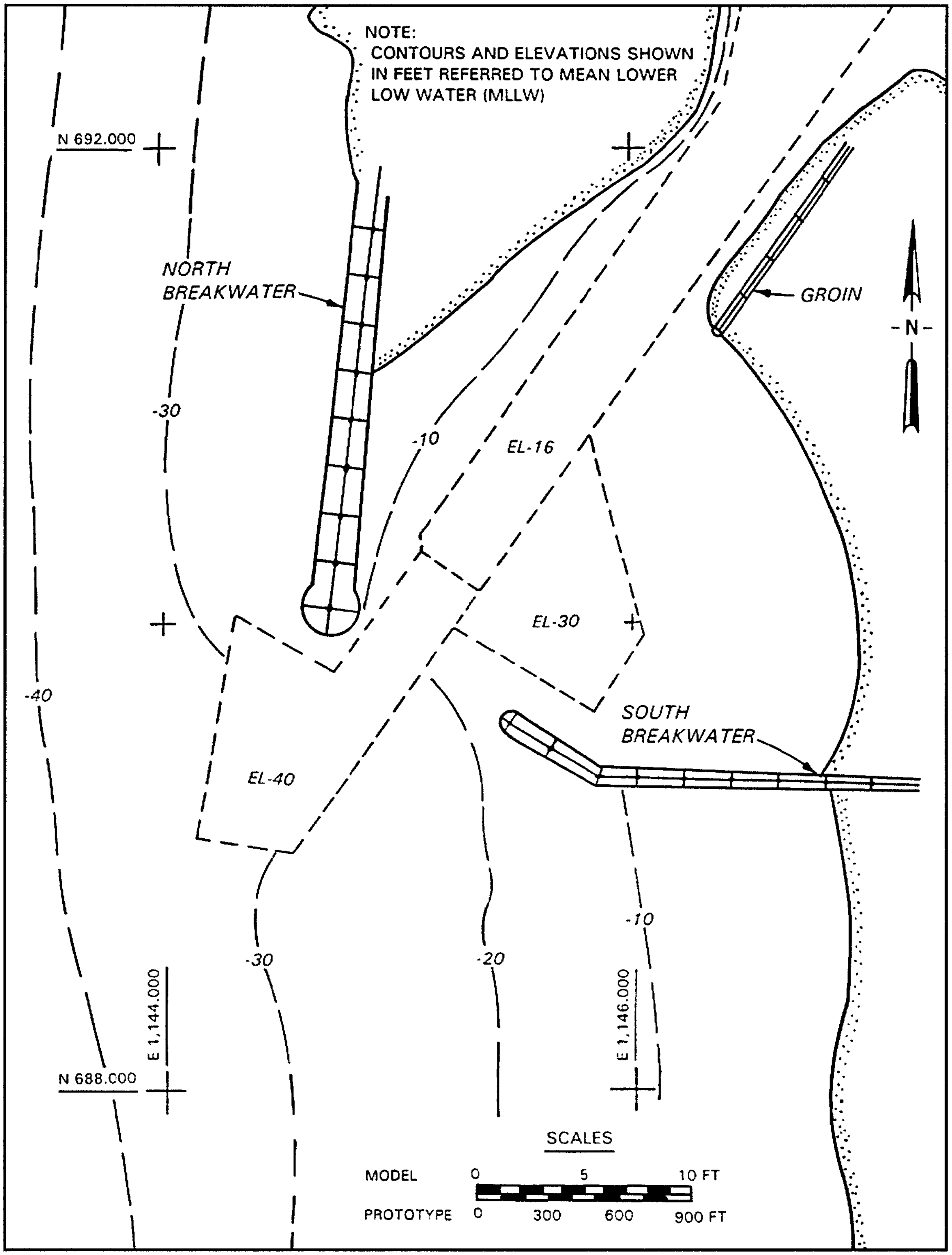

Figure 4. Plan view of 1995 entrance channel improvements

transport in the vicinity of Morro Bay entrance. The field study was conducted during the period November 1990 - March 1991 (Kaihatu, Andrassy, and Thompson 1992).

Numerical models

Numerical studies conducted at ERDC included the use of the Regional Coastal Processes WAVE Propagation model (RCPWAVE) (Ebersole 1985) to transform deepwater waves into breaking conditions at the site. Resulting breaking wave heights, periods, and directions were used to calculate longshore flux factors for use in sediment transport computations utilizing techniques found in the *Shore Protection Manual* (1984). Results indicated that 163,930 cu m (214,400 cu yd) of material would move toward the harbor entrance annually. This includes 12,770 cu m (16,700 cu yd) from the north beach and 151,160 cu m (197,700 cu yd) from the south beach.

Longshore sediment transport rates toward the harbor entrance were estimated by the Los Angeles District to be 361,700 cu m (473,000 cu yd) annually, including 54,300 cu m (71,000 cu yd) from the north and 307,400 cu m (402,000 cu yd) from the south. The Los Angeles District used the same modeling procedures as ERDC; however, different selections were made about grid cells contributing to transport toward the entrance. The difference between the Los Angeles District and ERDC estimates illustrates the difficulty in modeling this complex area. The Los Angeles District estimated that about 186,600 cu m (244,000 cu yd) of material will bypass the channel and that about 175,100 cu m (229,000 cu yd) would be trapped annually.

During maintenance dredging in the fall of 1990, the Los Angeles District dredged a test trench to assess shoaling rates in the area of the proposed project (Kaihatu, Andrassy, and Thompson 1992). The trench was 30 m (100 ft) wide and -10 m (-33 ft) deep and located within the existing entrance channel. Volumetric changes within the test trench as well as wave data at the site were obtained during a 4-month period. Shoaling rates correlated well with local wave conditions. Increases in shoaling coincided with increases in wave energy. Extrapolated results over the entire, limited time frame indicated an annual shoaling rate of 143,000 cu m (187,000 cu yd). Dredging records from 1944 - 1990 revealed an average annual sedimentation rate of 88,700 cu m (116,000 cu yd) at Morro Bay. Annual sedimentation rates varied, however, from 48,200 cu m (63,000 cu yd) to 175,900 cu m (230,000 cu yd). Variations appeared to be attributed to yearly variations in wave climate as opposed to dredging intervals.

A set of representative wave conditions was input into the ERDC numerical model Harbor, Deep Water (HARBD). This model was used to determine wave action near the entrance and inside the outer harbor for both existing and improved conditions (Kaihatu, Lillycrop, and Thompson 1989). The output of HARBD was then used to estimate breaking wave conditions through the channel. The numerical model indicated that the number of breaking days at the entrance annually would be reduced from 28 days to 2.5 days with the improvements. It also indicated, however, that wave breaking would move inside the outer harbor for the improvement plan and increase breaking days annually from 0.34 days to 5.03 days at that location. At the conclusion of the numerical investigation, a physical model study was recommended to gain an accurate prediction of absolute wave heights in the harbor entrance and broken wave propagation through the proposed dredged channel configurations.

Physical model

A 1:90-scale, three-dimensional hydraulic model of Morro Bay Harbor entrance was constructed and tested at ERDC to investigate the design of proposed channel depth modifications to improve navigation conditions and reduce maintenance dredging costs (Bottin 1993). Representative wave conditions as well as ebb tidal currents were reproduced in the model. The impact that proposed depth changes may have on wave conditions at the existing harbor structures also was addressed, and sediment tracer patterns were obtained at the entrance. Results indicated that the initially proposed, deepened entrance channel was effective in reducing wave heights in the entrance; however, wave heights at the head of the south breakwater significantly increased. The deepened entrance allowed more energy to reach the structure, as opposed to breaking and losing energy as with the existing contours. After studying numerous configurations, an optimum channel configuration was selected that resulted in improved navigation conditions and had no negative impacts on the existing structures. An additional sand trap area also was recommended north of the head of the south breakwater based on sediment tracer experiments. The configuration recommended in the physical model investigation was that constructed in the prototype in December 1995 (Figure 4).

Summary of predicted design performance

In summary, predicted performance of the improvement plan with respect to hydrodynamic conditions in the entrance and impacts to adjacent areas was defined by the three-dimensional (3-D) model for a range of wave conditions and directions. The physical model also predicted sediment movement patterns and deposition in a qualitative sense. Historical records, field measurements, and numerical models were used to predict sedimentation rates. Although these rates varied significantly, a 3-year dredging frequency was predicted for the harbor entrance based primarily on engineering judgment.

2 Monitoring Program

Monitoring Plan

A monitoring plan was developed prior to monitoring the Morro Bay Harbor entrance channel. During the development of the monitoring plan, specific hypotheses to be tested were laid out. The hypotheses to be tested are as follows:

a. The improvements constructed at the Morro Bay entrance in December 1995 will result in significantly improved navigation conditions in the harbor entrance.

b. The improvements will have no negative impact on the existing structures.

c. Improvements can be effectively maintained with a 3-year dredging interval in the entrance.

d. The model investigations accurately quantified wave conditions in the entrance and correctly defined sediment patterns and deposition in a qualitative sense.

e. Methodology used in determining sedimentation rates in the harbor entrance was valid based on field data, model predictions, and sound engineering judgment.

The objective of the monitoring was to determine if the nonstructural modifications at the harbor entrance were performing as predicted. Evaluation of hydrodynamic conditions and sedimentation rates in the harbor entrance as well as validation of models used as design tools were to be performed. Wave data (both inside and outside the harbor entrance), tidal elevations and currents, and bathymetry were to be obtained to determine design effectiveness of the harbor entrance alternative. Limited ground-based surveys and photogrammetric flights of the existing south breakwater were to be performed to determine if any negative impacts had occurred as a result of the dredging improvements. Data results obtained were expected to be used to study similar problems at other site-specific locations as well as for research and development studies. Both the field and laboratory were expected to gain from the monitoring effort.

Elements of the monitoring plan were to include data collection of waves, tidal elevations and currents, bathymetry, and ground and photogrammetric surveys of the south breakwater. More detailed information relative to the elements of the monitoring plan is provided in the following subparagraphs.

Wave data

A directional wave gauge was to be deployed outside the harbor in a depth of -14.3 m (-47 ft). This water depth is representative of the depth at which incident waves were specified for both numerical and physical model studies. Based on a wave refraction analysis (RCPWAVE), deepwater wave heights were propagated to an area immediately seaward of the harbor entrance at the -14.3 m- (-47 ft-) contour in the numerical model study and then used as incident waves for the detailed harbor grid. Incident wave conditions for the 3-D physical model also were defined at this depth. Two nondirectional pressure wave gauges were to be installed in the harbor entrance (one in the outer entrance and one in the inner entrance).

Comparison of model and prototype wave data

Prototype wave measurements were to be compared with those obtained in both the physical model and the numerical HARBD model. This data set would provide a unique opportunity in which to validate and/or improve our modeling technology.

Tidal data

A tide gauge was to be installed in the inner entrance to the bay, and colocated with the inner entrance pressure gauge, to obtain tidal elevations. These data were to be correlated with wave conditions in the entrance. They also would provide a complete, accurate data set of tidal elevations for Morro Bay.

Current data

An Acoustic Doppler Current Profiler (ADCP) was to be deployed inside the harbor entrance to collect currents along the channel cross section. The ADCP was to operate for approximately 2 weeks and obtain an adequate record of tidal currents. These data would be useful for evaluating scour/shoaling potential in the navigation channel and entrance for assessing wave-current interactions near the entrance, including wave breaking.

Bathymetric data

Bathymetric data obtained subsequent to dredging of the improvements at the Morro Bay entrance were to be analyzed to determine sedimentation since construction. During the monitoring effort bathymetry was to be obtained three times per year. The MCNP Program would fund two of these surveys and the Los Angeles District would fund one as part of their condition survey. Sedimentation rates would be calculated after each survey and correlated with prototype wave conditions obtained. The frequency of these surveys would provide very accurate seasonal data. These detailed data would provide a unique opportunity in which to validate/refine/improve sedimentation prediction technologies.

Ground-based survey and photogrammetry

Targets were to be established along the crest of the south breakwater during ground surveys and their x, y, and z coordinates would be obtained for verification of subsequent aerial photography. Stereopair photographs then would be obtained. From the photographs, a rectified map of the structure would be developed using 3-D stereoplotter, and digital orthophotos would be be prepared. From these products, contours of the breakwater would be generated. Photogrammetric analyses would be performed during the first and last year of the monitoring effort to determine if movement of armor units above the waterline had occurred. This element of the monitoring program would determine if the improvement plan had any negative impacts on the breakwater. It would also establish a base from which to evaluate the long-term structural stability of the structure.

Equipment and Data Collection

Monitoring of Morro Bay Harbor, CA, was conducted during the period January 1998 through August 2001. In general, most of the elements of the monitoring plan were completed as proposed. However, changes in procedures, techniques, etc. were made in some cases during actual monitoring. Actual elements of the monitoring program included prototype wave gauging, comparison of model and prototype wave data, collection of current and tide data, bathymetric analysis, and photogrammetric analysis of the south breakwater.

Prototype wave and current gauge locations are shown in Figure 5. Detailed information about this part of the monitoring program is given in a separate report (Garcia 2001). Overall data availability from the monitoring program is summarized in Figure 6. Availability of bathymetric survey data after initial construction of entrance channel improvements and prior to initiation of the monitoring program is also included. Equipment and methodology used during data collection are presented in the following subsections.

Prototype wave gauging

Prototype wave gauges were installed at Morro Bay on 11 September 1998. They consisted of a directional gauge outside the harbor (designated CA002), a nondirectional pressure gauge inside the harbor entrance (designated CA001), and a nondirectional buoy in the exposed harbor entrance (Figure 5). Data availability is summarized in Figure 6.

The directional gauge, CA002, was a short-baseline pressure gauge array (Howell 1998), deployed in a water depth of 14.3 m (47 ft). The gauge was bottom-mounted. Data were collected for 2,048 sec every 2 hr and stored internally. In this mode of operation, the gauge capacity is sufficient to store 12 months of data. The gauge was serviced four times during the 2-year

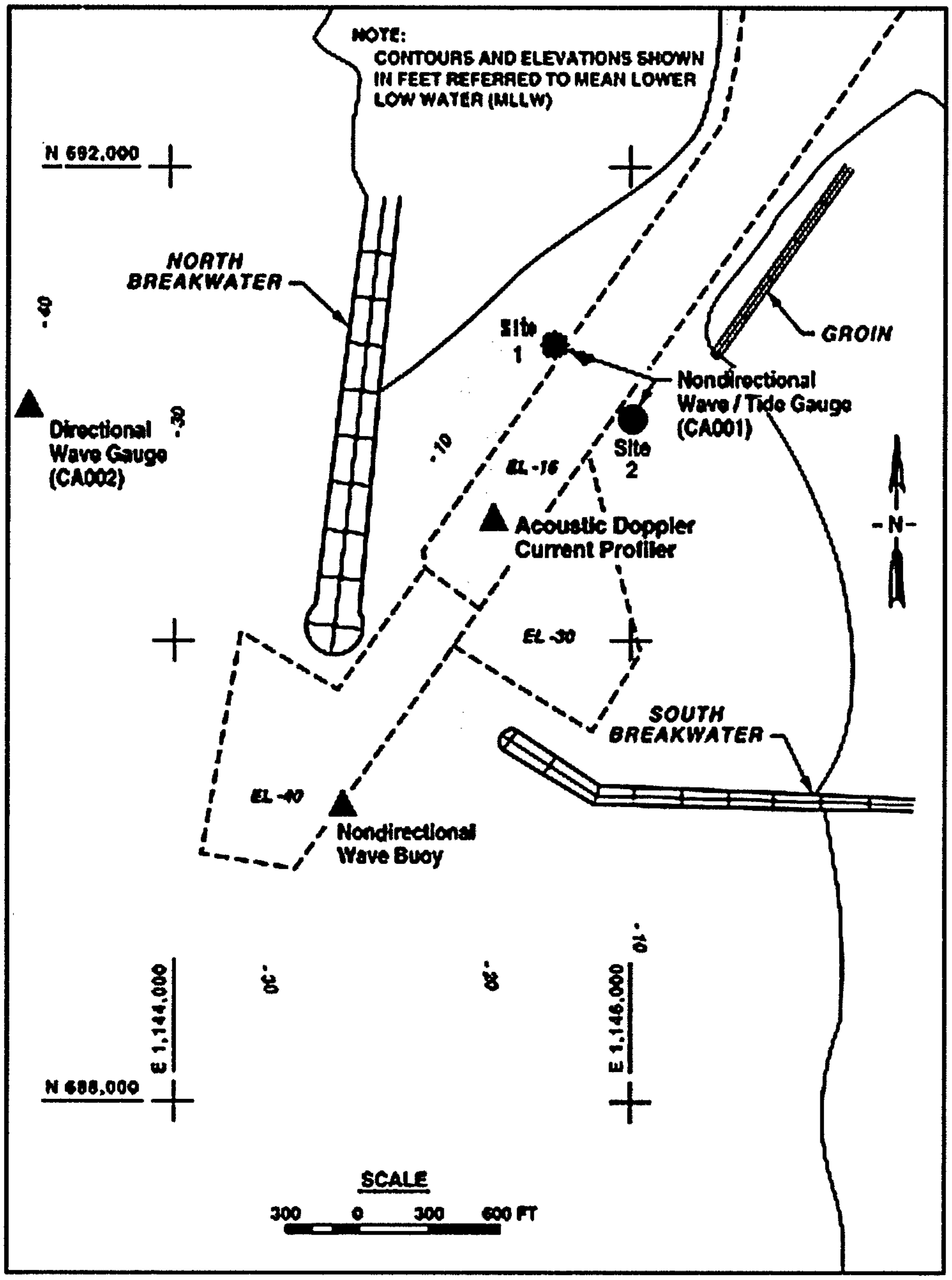

Figure 5. Prototype gauge locations at Morro Bay

monitoring period. Unfortunately, the gauge did not perform as well as expected. Data recording stopped on 22 November 1998 during the onset of a winter storm. The gauge was reactivated during a service visit on 24 March 1999. Directional data ceased on 8 June 1999, but nondirectional data continued until the gauge failed during a major storm on 28 October 1999. Nondirectional data recording resumed after a service visit on 6 April 2000 and continued until 29 August 2000, when the gauging program ended.

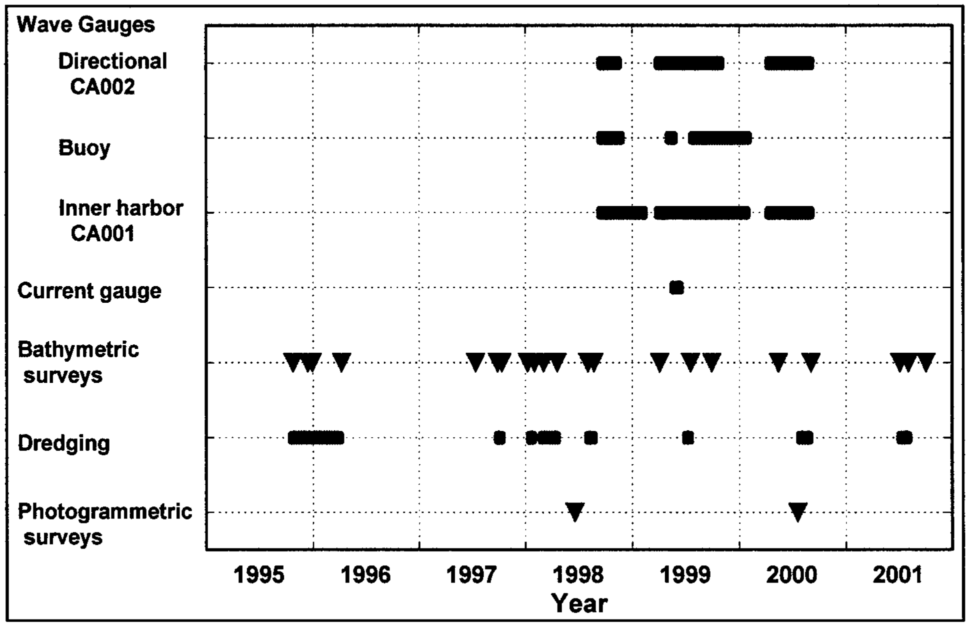

Figure 6. Summary of data availability

The nondirectional gauge inside the harbor entrance, CA001, was a single bottom-mounted pressure gauge. As with Gauge CA002, it was installed and operated by ERDC personnel. Wave data were recorded internally on the same schedule as Gauge CA002. Initial deployment was along the western edge of the navigation channel, in 6.1-m (20-ft) depth (Site 1 in Figure 5). Data from the initial deployment are available from 11 September 1998 to 8 February 1999. The gauge was reactivated on 24 March 1999 and continued collecting data until 25 January 2000. The gauge was moved to Site 2 (7.9-m or 26-ft depth) and reactivated on 6 April 2000 and continued to the end of the gauging program, 29 August 2000.

The nondirectional buoy was a Waverider accelerometer buoy at the landward edge of the deepened, exposed entrance. The buoy gauge was chosen because a bottom-mounted gauge at this location would be overly vulnerable to vessel traffic and bottom sediment movement. The buoy transmitted data in real-time to a receiver placed in a nearby U.S. Coast Guard office. Buoy data were processed by a contractor to give wave parameters every 20 min. The gauge failed on 22 November 1998, during the same storm that affected Gauge CA002. The gauge was reactivated and operated during the brief period 1-20 May 1999. It was reactivated on 19 July 99 and continued until an intense storm on 31 January 2000.

Tide and current data collection

Tidal elevation data were collected using the pressure wave gauge inside the harbor entrance, CA001. A water level data point was recorded every 3.75 min over the entire period of gauge operation.

Tidal current data were collected with an ADCP gauge during 17 May to 9 June 1999, a time period covering both spring and neap tide conditions. The gauge was located inside the entrance, near the inner pressure gauge, CA001 (Figure 5). Actual water depth was 6.7 m (22 ft). The gauge recorded data for 30 sec every 6 min, giving current speed and direction through the water column. Maximum ebb currents ranged from about 80 to 160 cm/s (2.6 to 5.2 fps) for neap and spring tide conditions. Maximum flood currents ranged from about 70 to 80 cm/s (2.3 to 2.6 fps).

Bathymetry

Bathymetry surveys of the Morro Bay project and adjacent areas were conducted 11 times between January 1998 and August 2001. An additional seven surveys were conducted prior to the start of the MCNP monitoring project. Bathymetric survey dates and coverage areas are given in Table 1. Contour plots of all survey results are given in Appendix A. Dredging activities at Morro Bay, including dates, coverage areas, and material quantities removed, are summarized in Table 2.

Table 1
Bathymetric Survey Dates and Coverage

Dates	Coverage	Remarks
19-22 Sept. 95	Entire harbor	Los Angeles District predredge survey
10 Dec. 95	Modified entrance channel and transition area	Contractor postdredge survey
26 Dec. 95	Modified entrance channel and transition area	Contractor postdredge, post-storm survey
4 Apr. 96	Entire harbor	Los Angeles District postdredge survey
8-11 July 97	Entire harbor	Los Angeles District condition survey
24 Sept. 97	Modified entrance channel and transition area	Los Angeles District predredge survey
8-9 Oct. 97	Entire harbor	Los Angeles District postdredge survey
2-6 Jan. 98	Entire harbor	Los Angeles District predredge survey
29 Jan. 98	Entire harbor	Contractor postdredge survey
27 Feb. 98	Entire harbor	Contractor predredge survey
7-8, 21 Apr. 98	Entire harbor	Los Angeles District postdredge survey
28-29 July 98	Modified entrance channel, transition area, main channel, and sand trap	Los Angeles District predredge survey
19-20 Aug. 98	Modified entrance channel, transition area, main channel, and sand trap	Los Angeles District postdredge survey
30 Mar., 12-14 Apr. 99	Entire harbor	Los Angeles District condition survey
13-14 July 99	Entire harbor	Los Angeles District postdredge survey
21-22 Sept., 28-29 Sept. 99	Entire harbor	Los Angeles District condition survey
9-11 May 00	Entire harbor	Los Angeles District condition survey
28-31 Aug. 00	Entire harbor	Los Angeles District post-dredge survey
19-21 June 01	Modified entrance channel, transition area, main and Navy Channel, and sand trap	Los Angeles District predredge survey
1-2 Aug. 01	Modified entrance channel, transition area, main and Navy Channel, and sand trap	Los Angeles District postdredge survey
2-3 Oct. 01	Modified entrance channel, transition area, main and Navy Channel, and sand trap	Los Angeles District predredge survey

Table 2 Dredging Activities			
Dates	**Coverage**	**Material Removed, cu m**	**Remarks**
Oct. – 26 Dec. 95	Modified entrance and transition area	413,350	Contract initial deepening
Jan. – 1 Apr. 96	Transition area, main channel, Navy Channel, and sand trap	339,630	Contract initial deepening
24 Sept. – 9 Oct. 97	Modified entrance channel	46,000	Los Angeles District maintenance dredging
12-26 Jan. 98	Modified entrance channel, transition area, Navy Channel, and Morro Channel	47,245	Contract maintenance dredging
19 Feb. – Apr. 98	Modified entrance channel, transition area, main channel, and Navy Channel	395,955	Contract maintenance dredging
29 July – 20 Aug. 98	Modified entrance channel, transition area, and main channel	60,100	Los Angeles District maintenance dredging
27 June – 13 July 99	Modified entrance channel, transition area, and main channel	102,600	Los Angeles District maintenance dredging
23 July – 26 Aug. 00	Modified entrance channel, transition area, and main channel	180,950	Los Angeles District maintenance dredging
23 June – 20 July 01	Modified entrance channel, transition area, and main channel	138,000	Los Angeles District maintenance dredging

Photogrammetric surveys

The head of the south breakwater was in a rather deteriorated condition at the initiation of the monitoring effort. Photogrammetric surveys were originally scheduled for FY 98 and FY 01; however, funding became available for the Los Angeles District to rehabilitate the breakwater during the course of the monitoring study. The second survey; therefore, was moved to FY 00, prior to rehabilitation, to determine if changes had occurred to the structure due to wave action. An aerial view of the south breakwater in July 2000, prior to rehabilitation, is shown in Figure 7. Photogrammetric surveys, as well as limited ground surveys for control, were conducted for the outer 167-m (550-ft) length of the south breakwater during June 1998 and July 2000.

To establish control for the photogrammetric work, ground surveys were initiated from existing known monuments, which included National Geodetic Survey stations and two Corps of Engineers benchmarks (brass disks) that had been established on the breakwater. This was accomplished by the use of Leica Global Positioning System (GPS) equipment and electronic land surveying techniques. Monuments used for control are shown in Figure 8. In addition, targets were established at intervals of approximately 30 m (100 ft) along the sea side, harbor side, and approximate center of the breakwater. Each target was marked with a drill hole 0.64 cm (1/4-in) in diameter, and 0.64 cm (1/4-in) deep, and painted with a circular target to ensure visibility in aerial photography. They were electronically surveyed with a Wild T2000/D15 electronic total station to form control by which the accuracy of the photogrammetric survey work could be validated. Horizontal positions were based on the California State Plane Coordinate System and elevations were referenced to the mean lower low water (mllw) datum.

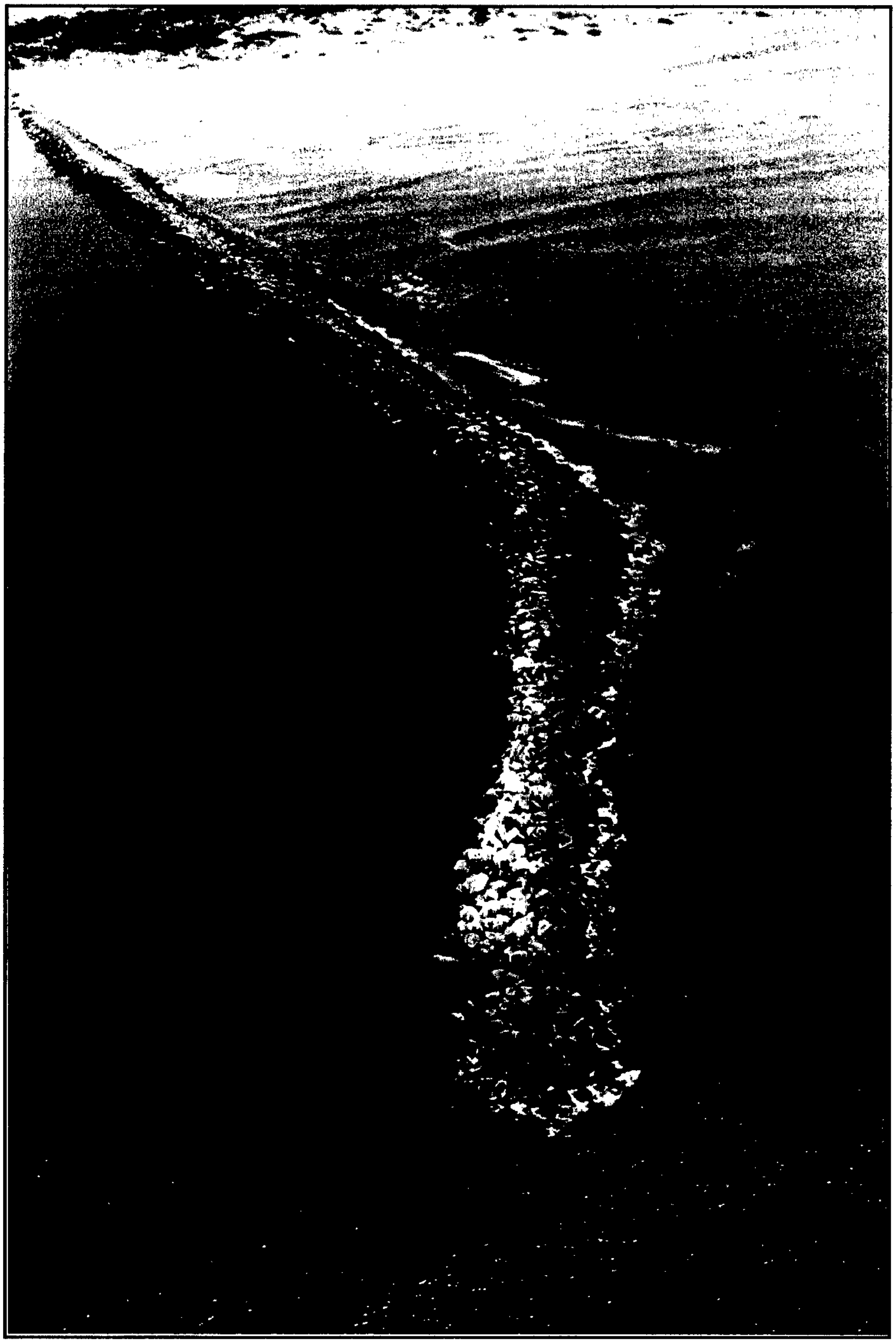

Figure 7. Aerial view of south breakwater

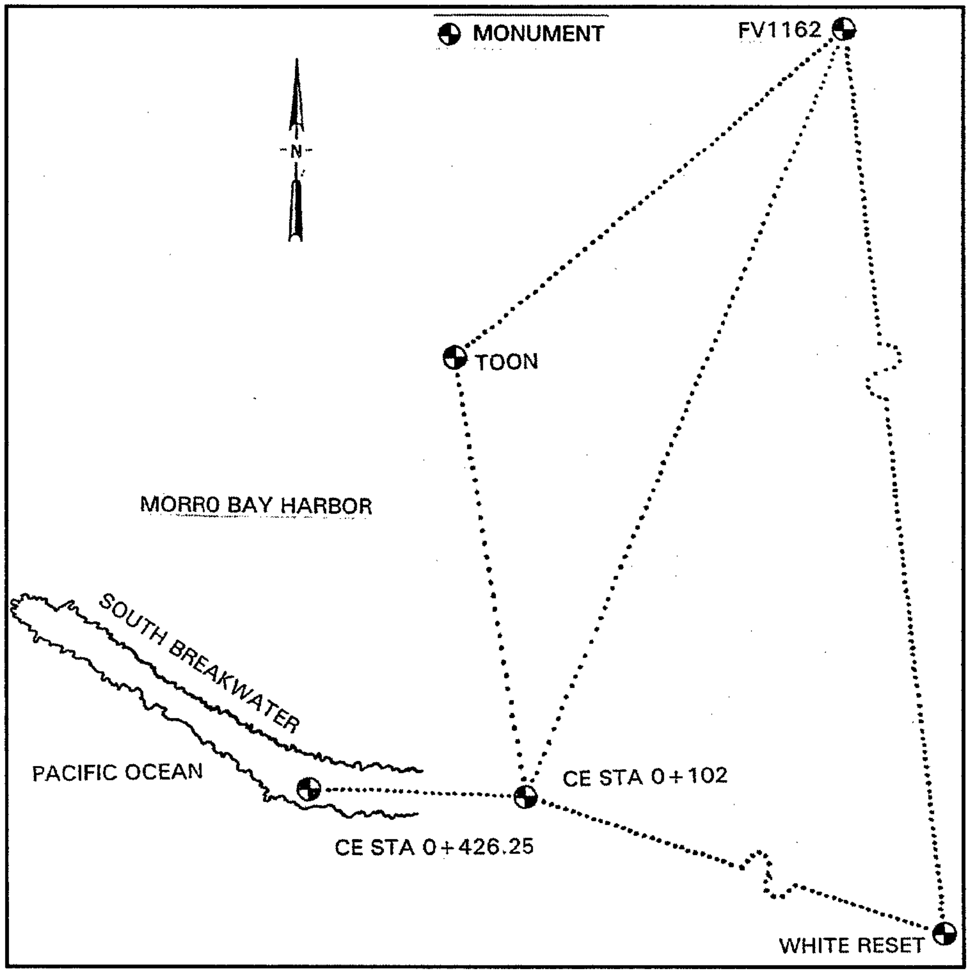

Figure 8. Monuments used to establish control for photogrammetric work

Aerial photography is an effective means of capturing images of large areas for later analysis, study, visual comparison to previous or subsequent photography, or measurement and mapping. Its chief attribute is the ability to freeze a moment in time, while capturing extensive detail. Low-altitude aerial photography was obtained along the breakwater with a Zeiss Jena LMK aerial mapping camera (22.9-cm by 22.9-cm (9-in. by 9-in.) format). The photos were secured from a helicopter flying at an altitude of 131 m (430 ft), which resulted in high resolution images and contact prints with scales of 1:860. Photographic stereo pairs were obtained during the flights.

When aerial photography is planned and conducted so that each photo image overlaps the next by 60 percent or more, the two photographs comprising the overlap area can be positioned under an instrument called a stereoscope and viewed in extremely sharp 3-D detail. If properly selected survey points on the ground have previously been targeted and are visible in the overlapping photography, very accurate measurements of any point appearing in the photographs can be obtained. This technique is called photogrammetry. The low-altitude stereo pair images obtained during aerial photography at Morro Bay Harbor south breakwater were viewed in a stereoscope and stereomodels were oriented to the monument and target data previously obtained. In the stereomodel, very accurate horizontal and vertical measurements can be made of any point on any armor stone appearing in the print. The stereomodel was used for all photogrammetric compilation and development of orthophotography.

Orthophotos combine the image characteristics of a photograph with the geometric qualities of a map. The digital orthophoto is created by scanning an aerial photograph with a precision image scanner. The scanned data file is digitally rectified to an orthographic projection by processing each image pixel. Orthophotos were prepared for the outer portion of the Morro Bay south breakwater. Precise horizontal measurements may be obtained from the orthophotos using an engineer scale since the image has been rectified and is free from skewness and distortion.

In addition to digital orthophotos, contour maps, and cross sections were developed for the outer portion of the south breakwater using digital terrain model (DTM). Maps consisted of an approximately 0.6-m (2-ft) grid pattern overlaid on the structure. Precise vertical and horizontal measurements were obtained at the intersections of the grid. Contour maps of the breakwater were developed from the DTM for a 0.3-m (1-ft) contour interval. In addition, using the analytical stereoplotter and DTM grid, cross sections of the breakwater were developed along the structure at 15.2-m (50-ft) intervals.

Data Results and Discussion

Wave gauge results – Morro Bay Gauge CA002

Gauge CA002 was located to provide directional data on wave conditions incident to the Morro Bay project. It collected a total of about 5 months of directional data and an additional 10 months of nondirectional data. An example plot of wave parameters from October 1998 is shown in Figure 9. The highest significant wave height during directional data collection was 3.14 m (10.3 ft), measured on 4 April 1999. Corresponding peak wave period and direction were 8.5 sec and 271 deg, respectively. Average wave parameters over all data collected are 1.1-m (3.6-ft) significant height, 11.6-sec peak period, and 265-deg direction. The larger wave heights, significant heights of 2 m (6.6 ft) or greater, were nearly all confined within a 22.5-deg direction band centered on 270 deg.

Outages of Gauge CA002 during the monitoring program and the absence of directional data during much of the program limited the usefulness of these data for defining incident wave conditions at the project. Limitations include no data during the months of December, January, and February (winter storm months), and no directional data coverage over any full intervals between bathymetric surveys.

Because these limitations seriously impact monitoring program objectives, other possible sources of incident wave data were reviewed to synthesize a more complete record of incident wave conditions during the program. Two

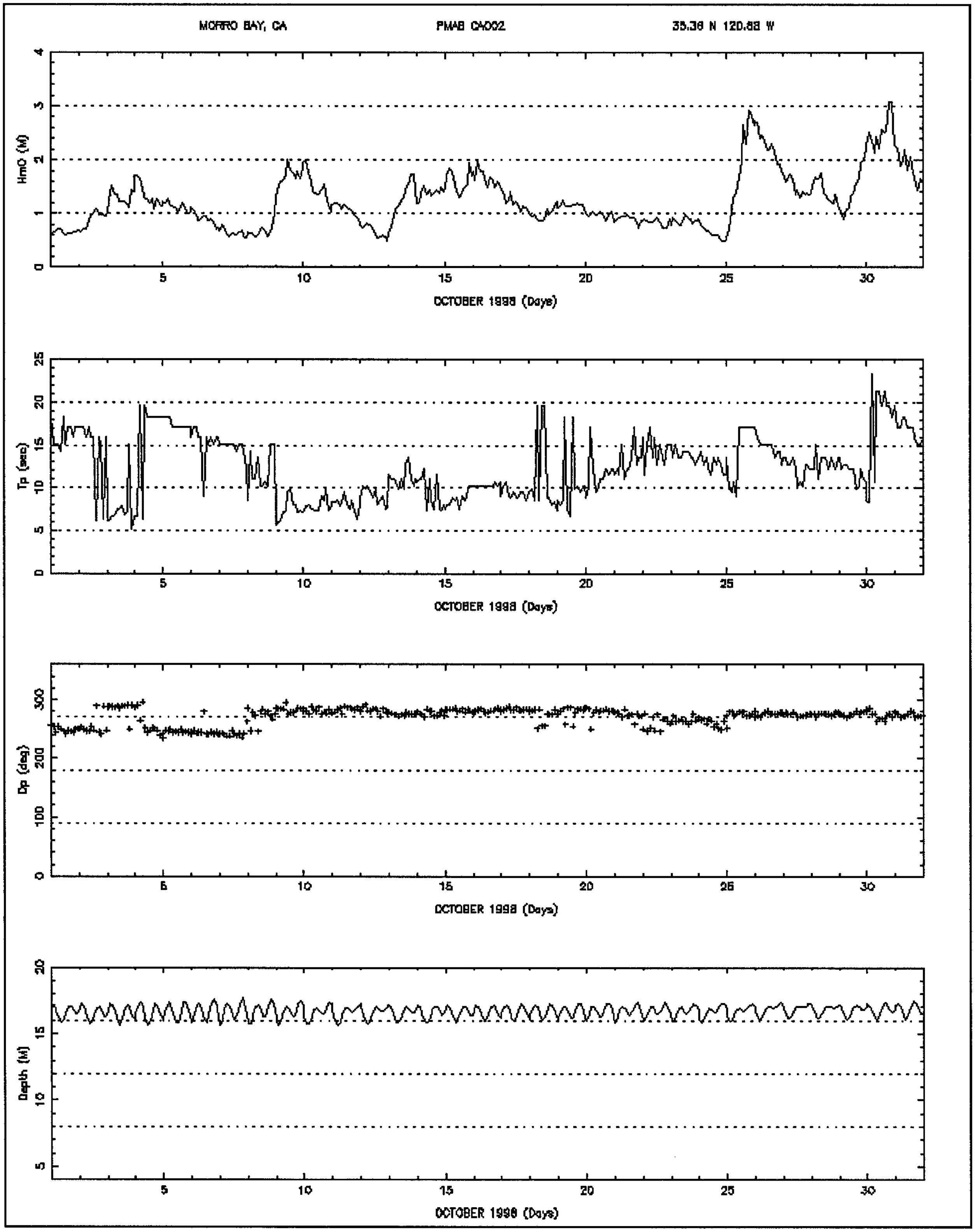

Figure 9. Example wave parameter time series, Morro Bay Gauge CA002, October 1998

consistently maintained offshore directional wave gauge sites are available within a reasonable distance from Morro Bay. North of the project site, the National Oceanic and Atmospheric Administration (NOAA) National Data Buoy Center (NDBC) operates a directional wave buoy near Monterey, CA, in water depth of 1,920 m (6,300 ft). South of the project site, the Scripps Institution of Oceanography (SIO) collects directional data at the Harvest Platform, a Texaco Oil Company oil-production facility in water depth of 204 m (670 ft). Two directional wave gauges operated at Harvest Platform within the monitoring program time period: a spatial array of pressure gauges, and an accelerometer buoy.

Both the Monterey buoy and Harvest Platform gauges provide directional wave data seaward of localized nearshore transformation effects. The Monterey buoy is 190 km (120 miles) northwest of Morro Bay and Harvest Platform is 110 km (70 miles) south-southeast. With the wind and wave climate characteristic to the California coast, especially the large spatial extent of major storms, it is reasonable to consider these gauges as possible sources of incident offshore waves at Morro Bay.

Wave gauge results – NDBC directional buoy

The NDBC directional wave buoy near Monterey, NDBC buoy 46042, is located at lat. 36.75 N, long. 122.41 W. The gauge has been collecting directional data since 1991, with occasional short gaps. A rose plot of significant wave heights during the years 1995-2000 shows wave climate characteristics (Figure 10). Waves generally come from directions between west and north-northwest. A secondary component of wave climate is evident from the south-southwest, but this component is overshadowed by the more commonly-occurring and typically more energetic waves from the northwest.

Waves recorded at NDBC buoy 46042 may be reasonably representative of deepwater offshore wave conditions at Morro Bay. However, they must be transformed into shallow nearshore waters representative of the entrance to Morro Bay Harbor before they can be considered comparable to data from the Morro Bay directional Gauge CA002. Bottom contours seaward of Gauge CA002 are sufficiently shallow to affect approaching waves and are reasonably straight and parallel. A standard wave transformation program based on a directionally-spread spectral wave condition propagating over straight, parallel bottom contours was applied (Jensen 1983; Gravens, Kraus, and Hansen 1991). The orthogonal orientation for determining local bottom contour alignment was taken as 266 deg azimuth. Wave energy directed offshore was removed. Also, a 50-deg arc at the north end of onshore directions was blocked to account for sheltering due to Pt. Estero and a 30-deg arc at the south end was blocked to account for sheltering by Pt. Buchon. Waves were transformed to a depth comparable to that of Gauge CA002.

The wave transformation program was applied to data from NDBC buoy 46042 during 1995-2000. Results were compared to Gauge CA002 during the times it was operational. An example comparison is shown in Figure 11. As in the example, the transformed NDBC buoy significant heights, peak periods, and

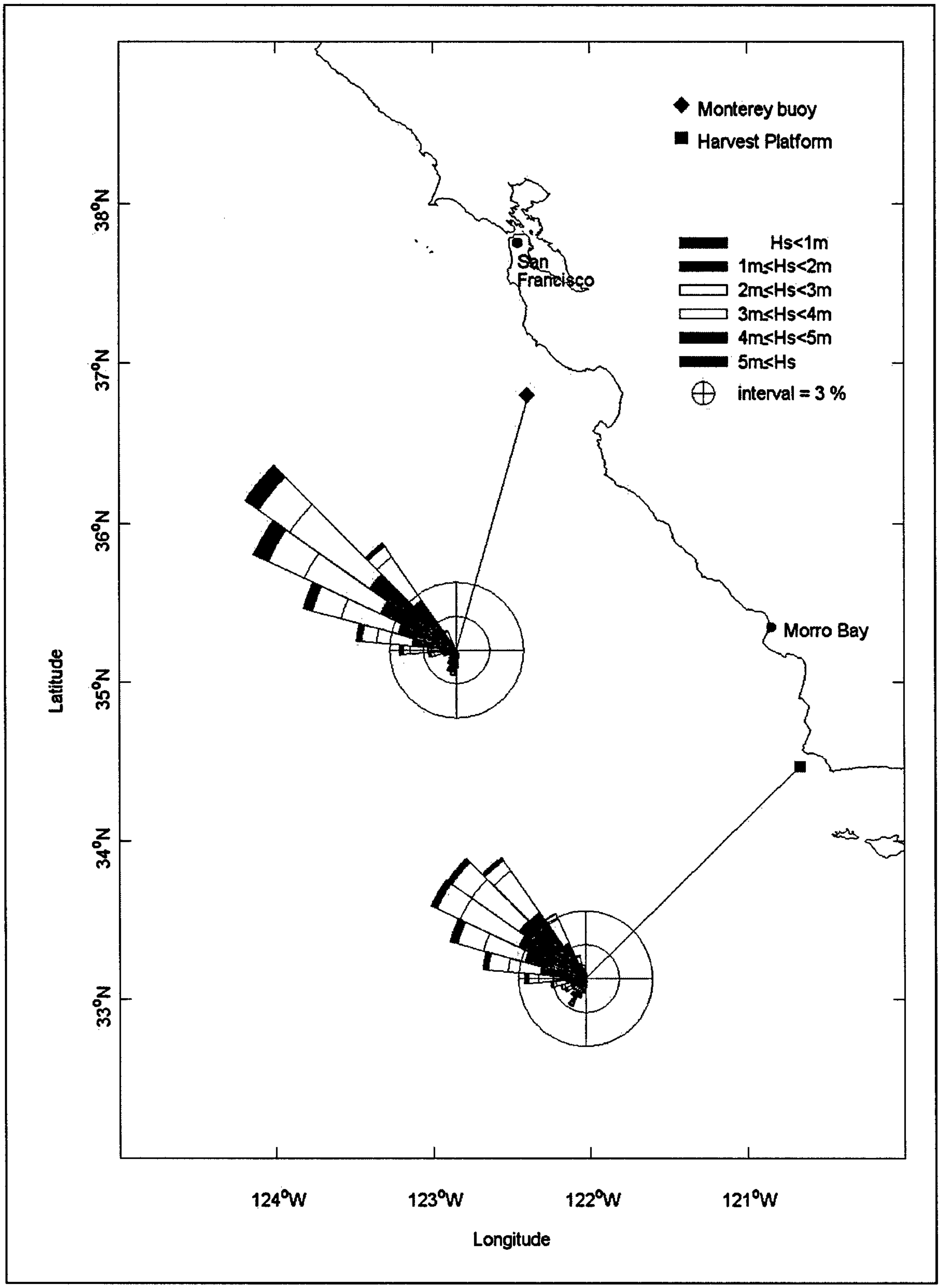

Figure 10. Wave roses of significant heights, NDBC Monterey buoy 1995-2000 and Harvest array 1993-1995

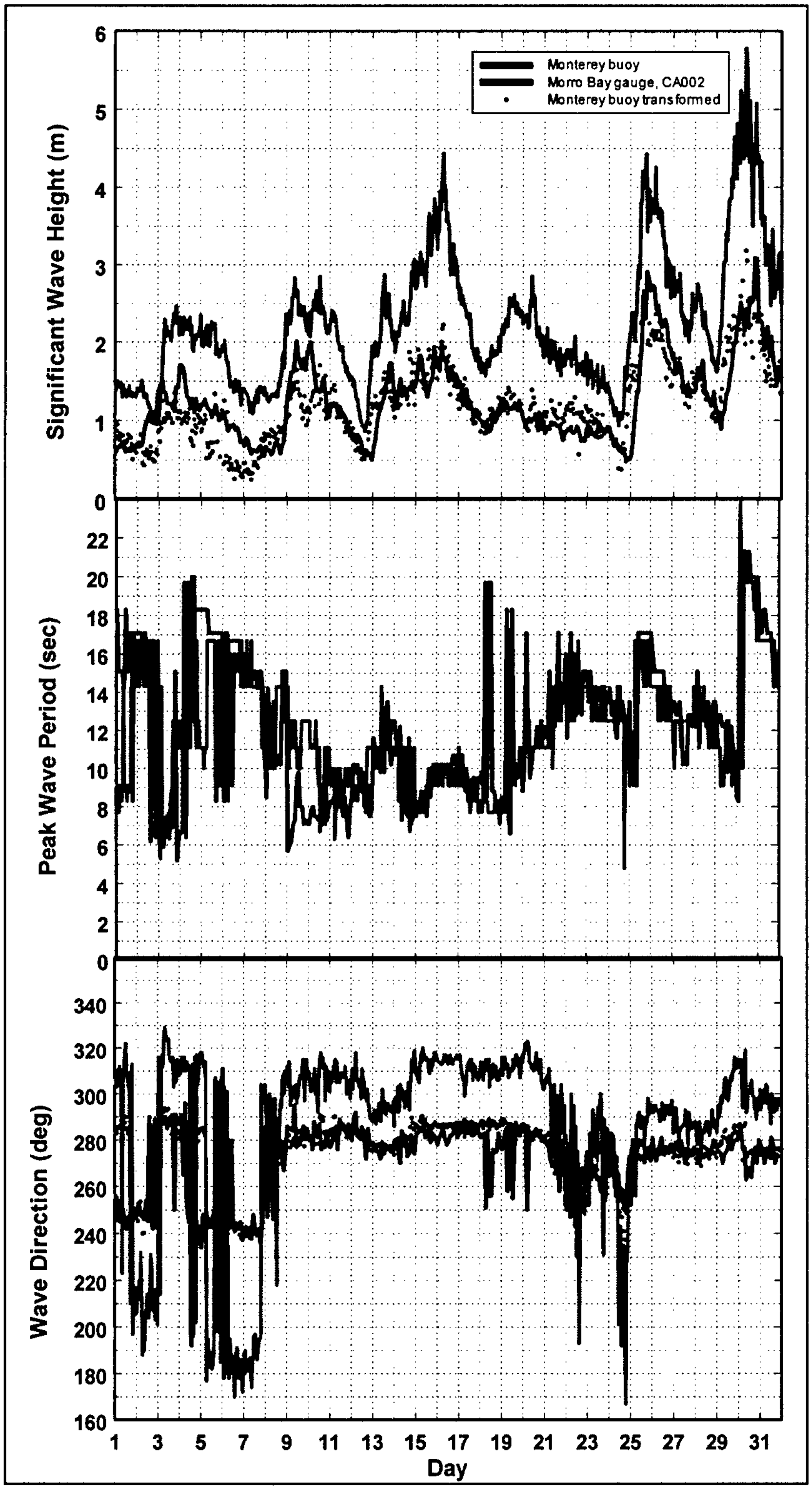

Figure 11. Comparison of wave parameters from NDBC Monterey buoy and Morro Bay Gauge CA002, Oct 1998

directions generally compare well with Gauge CA002 data. Based on these comparisons, the transformed NDBC buoy data were accepted as a reasonable auxiliary source of nearshore incident waves for Morro Bay Harbor.

Wave gauge results – Harvest Platform

The Harvest Platform gauges are located at lat. 34.48 N, long. 120.69 W. Gauge identifying numbers are 06301 for the array and 07101 for the buoy. The high resolution array collected directional data between November 1991 and January 1999, with several lengthy gaps in 1996 and 1997. The array was accidentally hit by a vessel on 28 March 1998 and reliable directional data are not available after that date. A rose plot of significant wave heights during the years 1993-1995 shows wave climate characteristics (Figure 10). As with NDBC buoy 46042, wave climate is dominated by waves from northwesterly directions, with a small secondary concentration of waves from south-southwest. In comparison to buoy 46042, the northwesterly component of wave climate at Harvest Platform is less energetic and the southerly component of wave climate is rotated slightly more toward the west. Overall, wave climate is remarkably similar at the two deepwater gauge locations. A detailed presentation of wave climate at Harvest Platform during 1993-1995 is given by Long (1998).

The directional buoy at Harvest Platform began operation in November 1995 and it is still operational. After a large gap from March 1996 until March 1998, the buoy has provided a consistent, reliable record of directional waves to augment and extend the array data.

Waves from Harvest Platform gauges must be transformed into shallow nearshore waters representative of the entrance to Morro Bay Harbor before they can be considered comparable to data from the Morro Bay directional Gauge CA002. The wave transformation approach used with NDBC buoy 46042 data was applied to data from the Harvest Platform gauges. An example comparison between Harvest Platform data and Gauge CA002 data is shown in Figure 12. Transformed significant heights, peak periods, and directions generally compare very well with Gauge CA002 data. They also compare well with transformed NDBC buoy 46042 data. As with the NDBC buoy data, the transformed Harvest Platform data were accepted as a reasonable auxiliary source of nearshore incident waves for Morro Bay Harbor.

Incident wave data summary during monitoring program

With directional wave data from NDBC buoy 46042 and Harvest Platform transformed to be representative of nearshore incident waves at Morro Bay Harbor, a continuous incident wave record can be reconstructed over the full time period since the harbor entrance was modified. Time-history of nearshore significant wave height from available sources is summarized in Figure 13. Bathymetric survey and dredging intervals are also shown. Survey intervals are numbered sequentially for reference in bathymetric change analysis. Some intervals, when entrance dredging was in progress, are not numbered, since they are not appropriate for bathymetric change analysis.

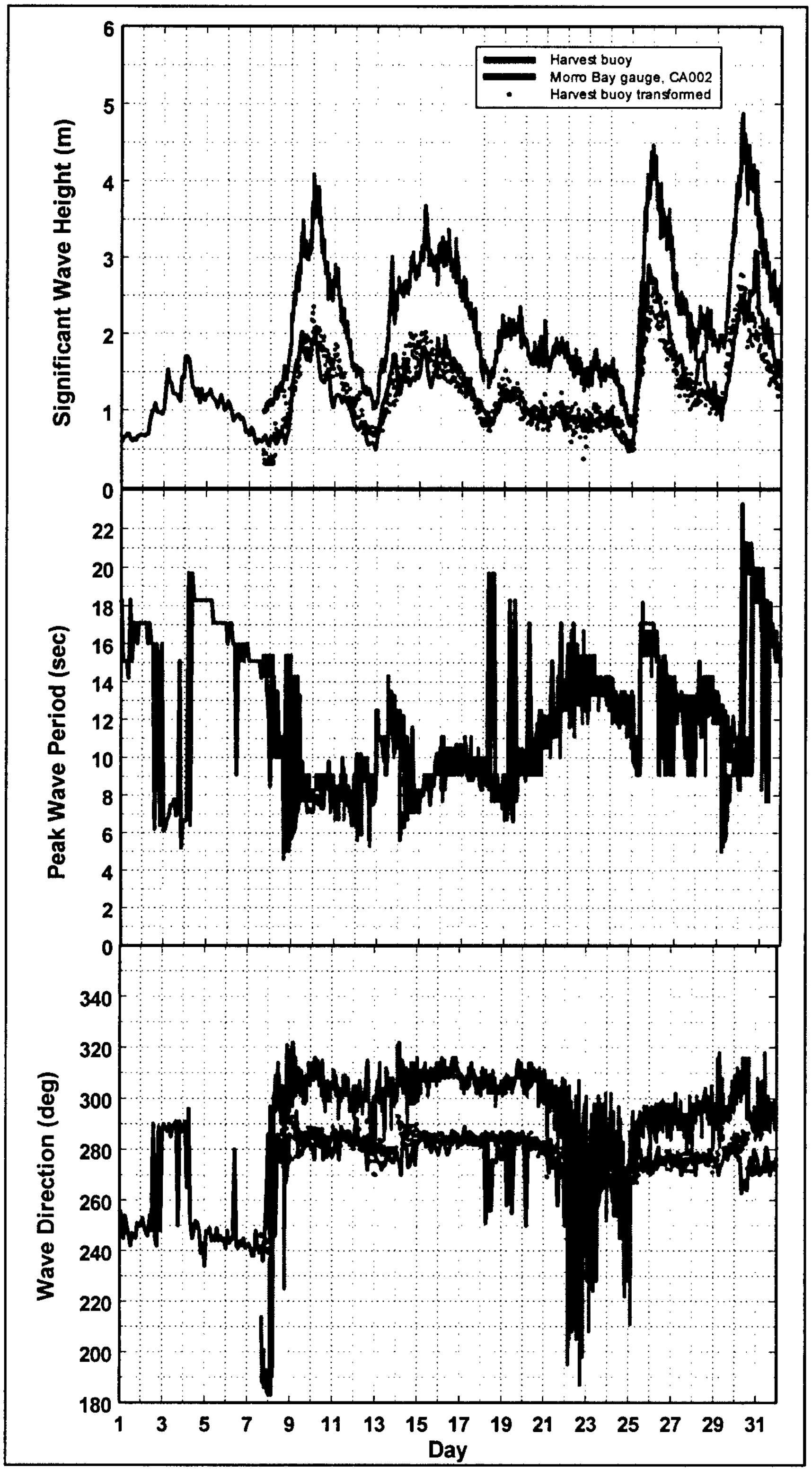

Figure 12. Comparison of wave parameters from Harvest gauges and Morro Bay Gauge CA002, October 1998

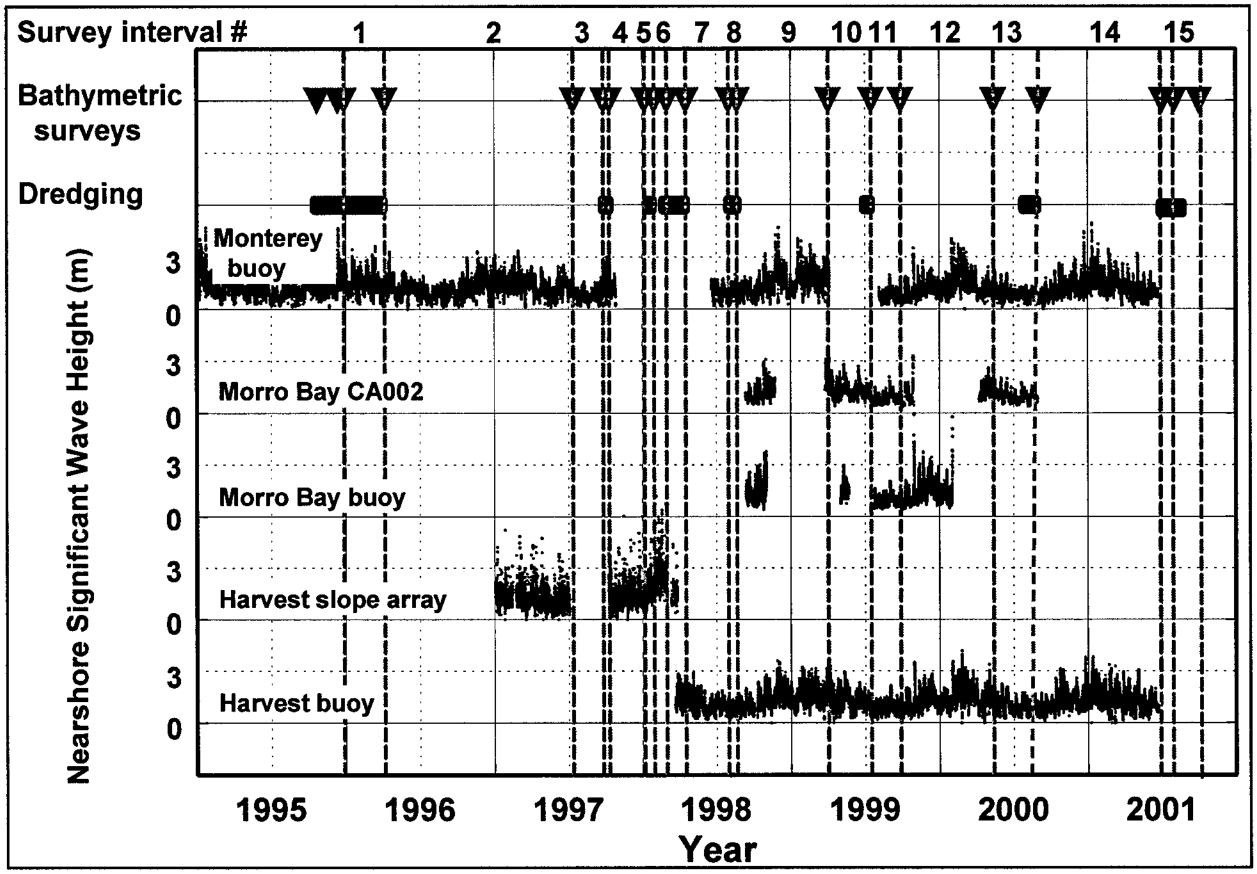

Figure 13. Time-history of nearshore significant wave heights from available sources, bathymetric survey intervals, and dredging activity; offshore sources are transformed to Morro Bay Harbor entrance (some survey intervals when dredging was in progress are not numbered since they are not appropriate for bathymetric change analysis)

Directional wave information over each numbered survey interval is needed in conjunction with bathymetric change analysis. The Morro Bay gauges do not provide adequate coverage of any intervals. Morro Bay Gauge CA002 provides significant wave height coverage of intervals 10 and 11, but directional data failed during interval 10 and was not restored. The Morro Bay buoy also provides significant height coverage of interval 11, but it, too, did not provide wave direction. Therefore, transformed data from offshore data sources is essential for bathymetric change analysis.

The transformed Monterey NDBC buoy data cover many of the survey intervals. However, both the array and buoy transformed data from Harvest Platform are needed to complete coverage of all survey intervals, as shown in the figure. Only selected segments of Harvest gauge data are shown, during periods of consistent operation and critical need to bridge gaps in the Monterey buoy data.

The figure provides a useful overview of wave conditions during the monitoring project. For example, wave heights were consistently low throughout interval 11 and intervals 9, 12 and 14 experienced several major storm time periods. For time periods when the Monterey and Harvest gauges are both available, consistency of results can be judged. When these sources north and south of Morro Bay agree, they can be expected to be representative of conditions at Morro Bay. When they differ, wave conditions at Morro Bay can generally be expected to be in between.

The Morro Bay buoy is noted to have several unusually high significant wave heights, up to 6 m (19.7 ft), during interval 12. Other sources indicate energetic storms at the same times, but significant heights are considerably lower. Contrary to other gauges, this buoy is in a location where waves may be strongly modified by rapid tidal currents through the breakwater gap, shallow bottom, and wave breaking. In highly energetic events, the buoy response to sea surface fluctuations may even be distorted in ways that are not compensated in the data analysis. Thus, data from this buoy should be treated as localized information.

Wave gauge results – Morro Bay Gauge CA001

The inner harbor gauge deployed under the Morro Bay monitoring program operated successfully during most of the program. It provided significant height and peak period wave parameters. The gauge was well protected from incident ocean waves and significant heights were generally very low. The second location for the gauge was more exposed than the initial location, but significant heights were still low. Typically, significant height was less than 0.2 m (0.7 ft). The maximum value during the monitoring program was 0.84 m (2.8 ft), recorded at 2000 Greenwich mean time (G.m.t.) 18 April 2000. Energetic events at Gauge CA001 appeared to be more related to local winds than to incident ocean wave conditions. Peak periods were usually representative of either incident ocean waves or much longer period oscillations affecting the semienclosed harbor area with comparable or greater energy than the residual ocean waves. Wave records collected at Gauge CA001 are not designed to give accurate data on long-period oscillations. However, oscillation data could be extracted from the water level data records collected at Gauge CA001, should that be desired in future studies. Detailed results from Gauge CA001 are given by Garcia (2001).

Comparison of prototype and physical model wave estimates

The modified entrance design for Morro Bay Harbor was based primarily on physical model experiments (Bottin 1993). Numerical model experiments also played a role in early phases of project development (Kaihatu, Lillycrop, and Thompson 1989). Both physical and numerical model studies included the transformation of incident waves over local entrance bathymetry, through the breakwater gap, and into the protected harbor area. One monitoring study objective was to use prototype data to evaluate the accuracy and effectiveness of the previous model studies. Comparison to physical model studies is considered in this section and numerical model studies are considered in the following section.

Directional Gauge CA002 is situated near the seaward boundary of the physical and numerical models. It serves as the incident wave condition. The nondirectional buoy and inner harbor Gauge CA001 provide wave data within the physical and numerical model domains.

Prototype cases were selected based on the following criteria at the outer gauge: significant wave height greater than or equal to 2 m (6.6 ft) (minimum significant height in physical model experiments was 2.4 m (7.9 ft)); peak period within 0.5 sec of a physical model experiment; peak wave direction within 5 deg of a physical model experiment. Data from the nondirectional buoy and inner harbor gauge for these cases were compared to corresponding physical and numerical model estimates, as discussed in the following paragraphs.

Since all of the selected prototype cases with concurrent data from both shoreward gauges were for incident wave directions of 275 deg, only the 275-deg physical model cases were considered for comparison. Additional criteria for selecting physical model cases were: Plan 14 configuration (matches prototype project), spectral experiments, and water level of 0.0 m. Physical model runs with 0.0-m water level did not include any tidal currents.

Comparisons are presented as wave height variation along the navigation channel center line. Distance along the channel center line is measured from a reference point seaward of the entrance (Figure 14). Prototype and physical model wave heights are converted to amplification factor by dividing channel wave heights by corresponding incident height. Comparison plots for peak periods of 12, 15, 17 and 20 sec are given in Figures 15-18. The prototype nondirectional buoy and inner harbor gauge data are shown as single points in each plot, located at distances of 400 m and 975 m, respectively. Each prototype point represents the average amplification factor for all matching prototype cases.

Wave height transformation along the physical model channel is generally consistent with the limited prototype data. At the nondirectional buoy location, the physical model shows wave heights comparable to the incident wave height or slightly lower. The prototype gauge shows wave heights 10-20 percent higher than incident. The discrepancy is probably due to three factors. First, the prototype gauge is located at the edge of the dredged flared entrance, south of the channel center line. Thus, it is further from the area where wave diffraction around the north breakwater head is affecting waves and it is in an area where waves may begin shoaling on the dredged slope. Second, bathymetry in the prototype changes with time and the entrance area was at least somewhat more shallow than the ideal project depths molded into the physical model. The prototype gauge is in an area particularly prone to shoaling. For the mild storm events captured in the prototype data, waves in the entrance would be pre-breaking and wave height would be expected to increase over shoaled entrance bathymetry. Third, tidal currents probably also influence prototype data. The gauge is in line with ebb current jets flowing out of the entrance gap. Interactions between ebb currents and incoming waves would tend to increase wave height in the prototype.

At the inner harbor gauge location, physical model wave heights tend to be comparable to or higher than the prototype data. Overall, the physical model effectively predicted decay of wave height between incidence and this sheltered

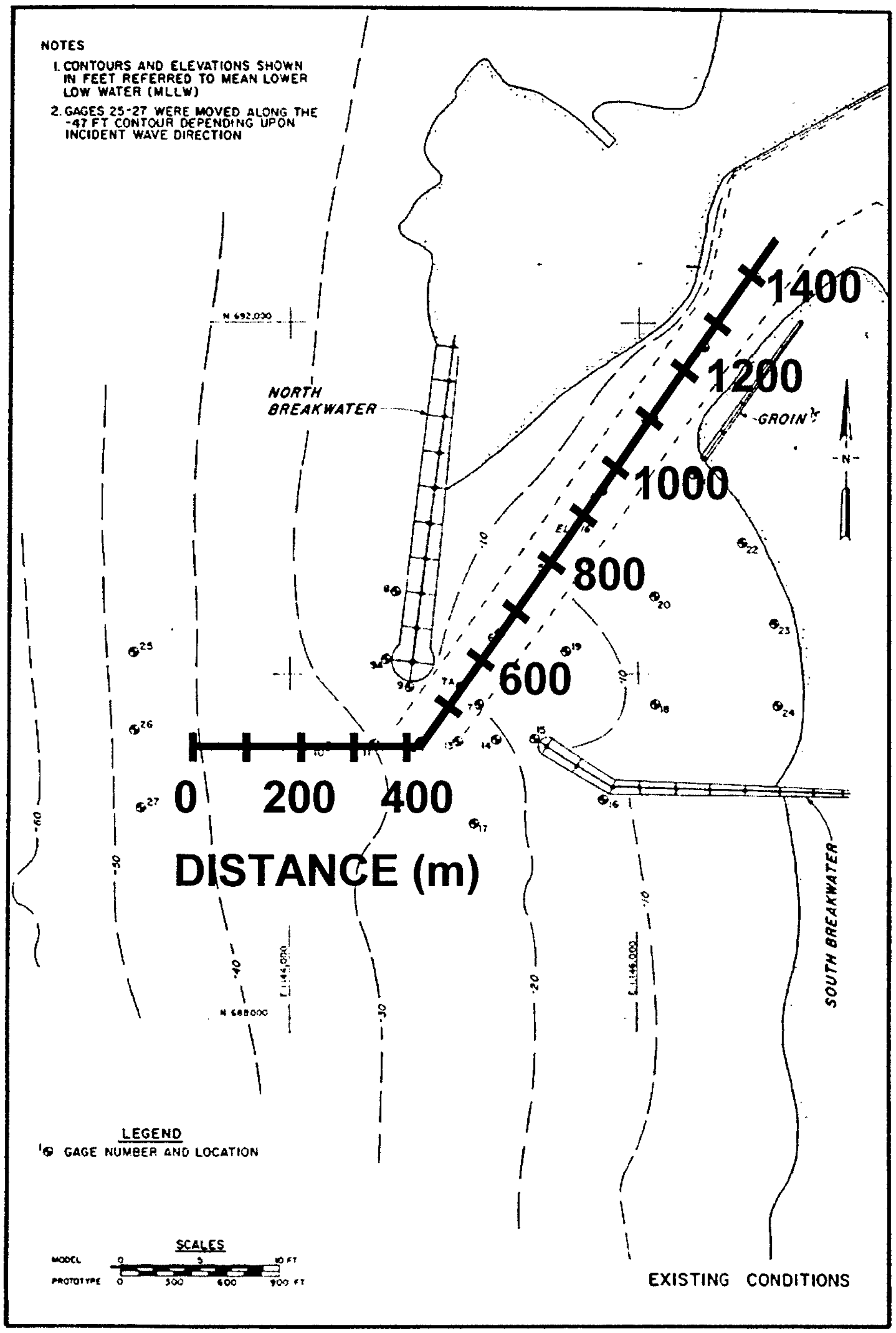

Figure 14. Reference distances along channel center line for model comparisons superimposed on physical model layout

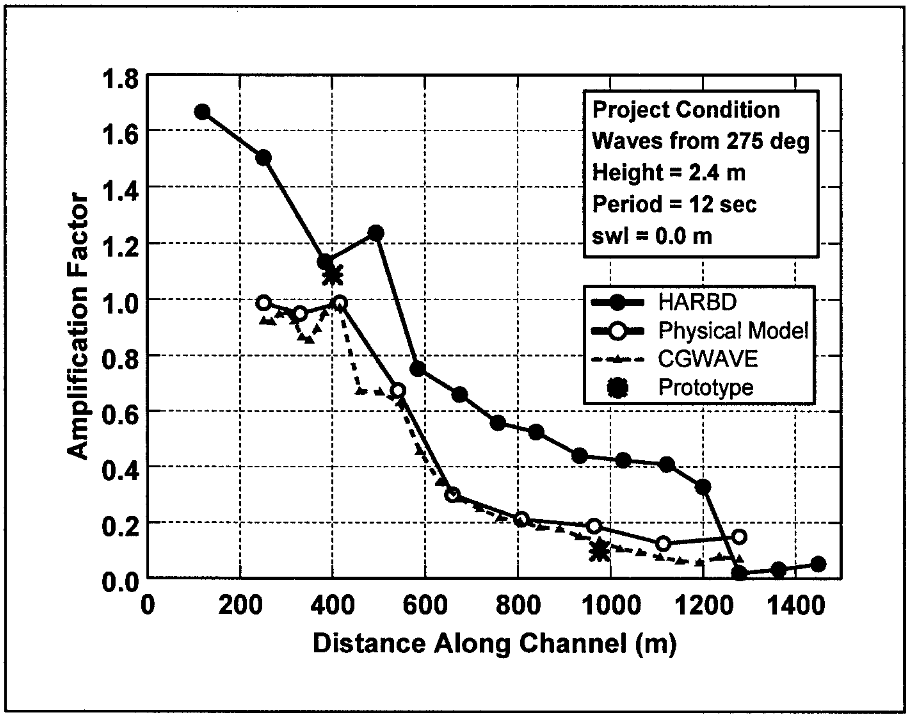

Figure 15. Comparison of model and prototype wave height amplification factor, waves from 275 deg azimuth, T = 12 sec

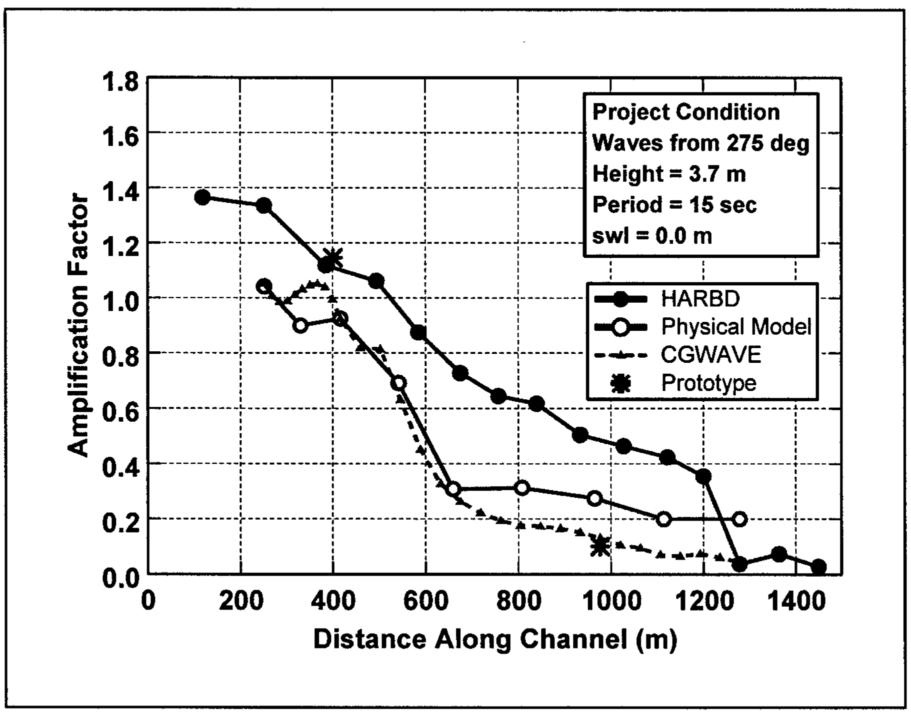

Figure 16. Comparison of model and prototype wave height amplification factor, waves from 275 deg azimuth, T = 15 sec

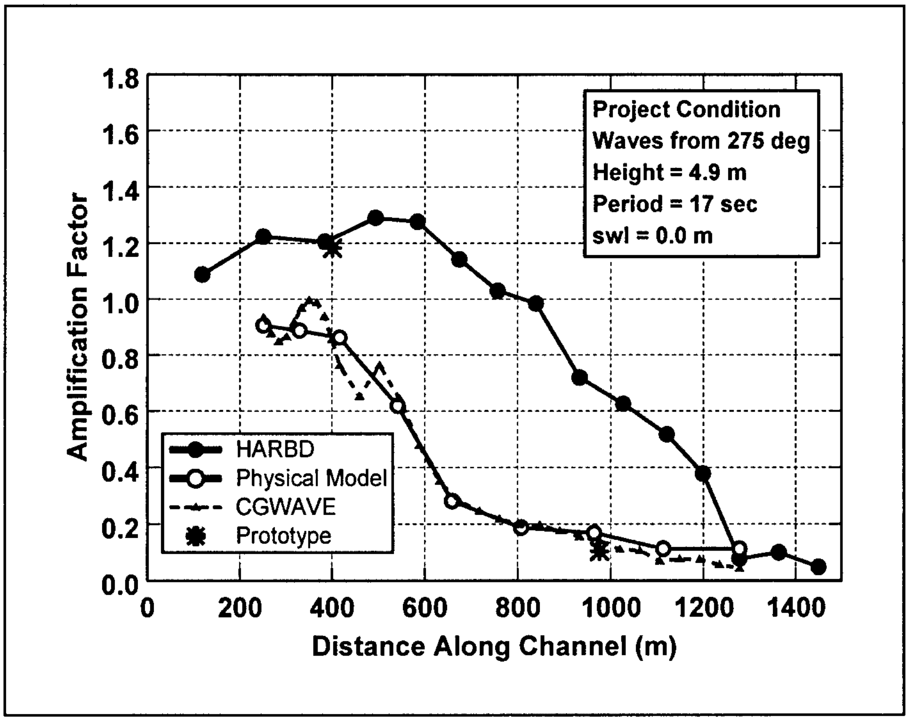

Figure 17. Comparison of model and prototype wave height amplification factor, waves from 275 deg azimuth, T = 17 sec

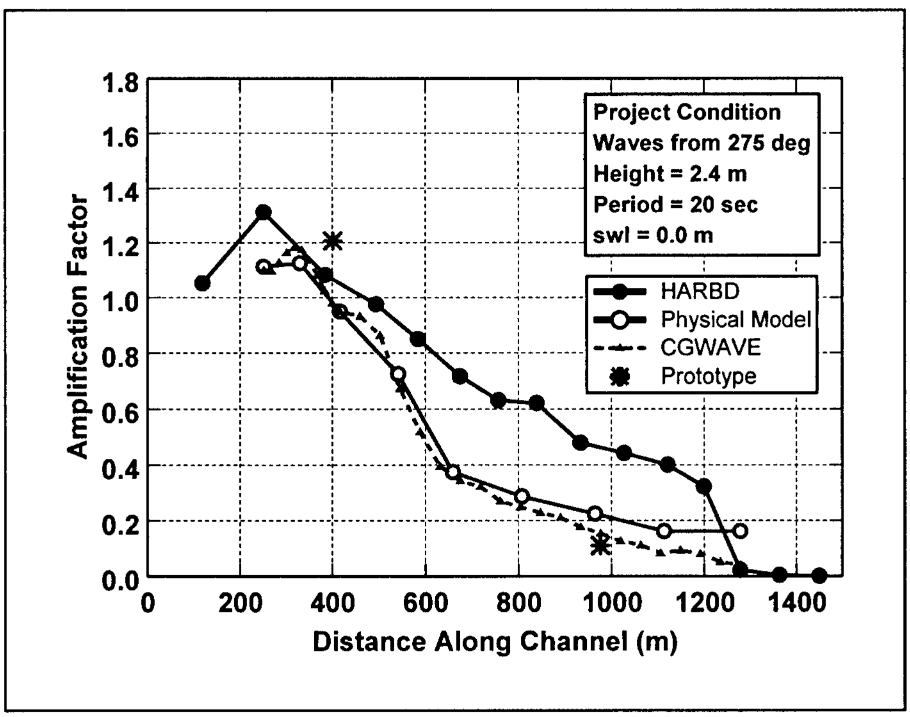

Figure 18. Comparison of model and prototype wave height amplification factor, waves from 275 deg azimuth, T = 20 sec

location. A cause of elevated heights in the physical model is the difference in reflectivity of model surfaces and boundaries compared to the prototype. The model bottom and shorelines are all rigid cement and they reflect wave energy more effectively than natural materials. Differences in gauge location also contribute to model/prototype differences. Physical model gauges were located along the channel center line. For practical reasons, the prototype inner harbor gauge was placed along the channel flank, in a more protected location than the channel center line in the initial deployment and in a more exposed location in the second deployment.

Comparison of prototype and numerical model wave estimates

Numerical model results are also shown in the comparison plots (Figures 15-18). The original HARBD results are for the wave period and direction at the HARBD model boundary best matching physical model incident wave parameters. Alternative 6 in the original HARBD study is used as a best match to the project condition. HARBD was run only for regular (monochromatic) waves.

Conjugate Gradient WAVE model (CGWAVE), the present CHL technology for numerical harbor wave modeling, was run for the four comparison cases as part of this monitoring study, and results are shown in the figures. CGWAVE runs were designed to match physical model experiments, including unidirectional, spectral waves, similar bathymetry, and wave breaking. The CGWAVE model domain extends significantly further seaward than the HARBD domain in previous studies.

HARBD results show a diminishing wave height as waves progress from incidence into the sheltered part of the channel. HARBD results are remarkably close to nondirectional buoy results, but considerably higher than inner harbor gauge results. Height amplification factors are greater for HARBD than for the physical model at all but the most inner end of the channel. HARBD, as applied in the original study, suffered from several major limitations, including regular (monochromatic) waves, no breaking, and restricted grid size and coverage area. The regular wave representation can lead to strong reflection patterns, wave heights significantly greater than incident wave height outside the harbor entrance (also evident in physical model data for regular waves, which is not shown here but included in Bottin 1993), and erratic wave height variations over short distances. The lack of wave breaking in HARBD is a serious limitation for the Morro Bay Harbor application which was not easily remedied.

CGWAVE results compare much more favorably than HARBD results with physical model data. This is partly attributable to CGWAVE being a more comprehensive model and partly to CGWAVE being expressly configured to match physical model test conditions. CGWAVE matches the inner harbor prototype gauge remarkably well. As with physical model data, it falls below the nondirectional buoy data, helping to support the explanation that currents, shoaling, and distance from the north breakwater head may be affecting the prototype data at this location. Some CGWAVE results show an oscillatory variation seaward of the breakwater gap. This variation disappeared in results from some additional CGWAVE runs with directionally spread, rather than unidirectional spectra (not shown). Prototype waves are directionally spread, and physical models may induce some directional spreading, even when wavemakers are run in a unidirectional mode.

More complete comparison plots of HARBD, CGWAVE, and physical model results are provided in Appendix B. Additional experimental cases are included for the project condition and similar comparisons for the preproject condition are shown. As in the figures, HARBD wave heights were generally higher than the physical model, especially inside the breakwater gap; and CGWAVE wave heights compared very well to the physical model.

Tide and current data results

Tidal elevation data were successfully collected at Gauge CA001 over nearly the full duration of the monitoring program gauge deployment, September 1998 to August 2000. These data are discussed and presented by Garcia (2001). Tidal current data collected during a special monitoring period are also presented and discussed by Garcia (2001).

A time-history of tidal elevation and currents during the 25-day monitoring period is shown in Figures 19-21. NOAA tidal elevation data from nearby Port San Luis, collected at 6-min intervals, is also shown in Figure 19. Spring tides at Morro Bay, occurring during the first few days of measurement, are characterized by a large diurnal inequality. During spring tide, higher high water is immediately followed by lower low water, giving a large elevation change over a relatively short time. Lower low water is followed by lower high water, a high tide with maximum elevation considerably reduced from the previous high tide. Lower high water is followed by higher low water, which is in turn followed by higher high water again.

The strongest entrance currents are expected around parts of the tidal cycle when elevation is changing most rapidly. Thus, the drop from higher high water to lower low water during spring tides coincides with the strongest currents measured at Morro Bay. These strongest currents are always ebb currents, flowing from the harbor out through the breakwater gap. Flood currents, flowing from the ocean into the harbor, and neap tide ebb currents can also be significant, but considerably weaker than maximum ebb currents during spring tide.

Strong currents in the Morro Bay Harbor modified entrance and transition areas will impact incoming waves, sediment transport, and shoaling patterns in the project area. To gain perspective on the relative impact waves and currents have on sediment movement in the modified entrance, the computer model WSTRANS was run for a range of conditions representative of the Morro Bay Harbor entrance (Wikramanayake and Madsen 1994). Sediment transport rate in 9.1-m (30-ft) water depth for wave heights up to 4 m (13 ft), wave periods of 6-20 sec in 2-sec intervals, and ebb currents up to 1.5 m/s (4.9 fps) are shown in Figure 22. Wave direction was from the west (270 deg) and current was aligned with the entrance channel inside the breakwater gap. Different wave periods are not distinguished in the figure, but the higher transport rates correspond to

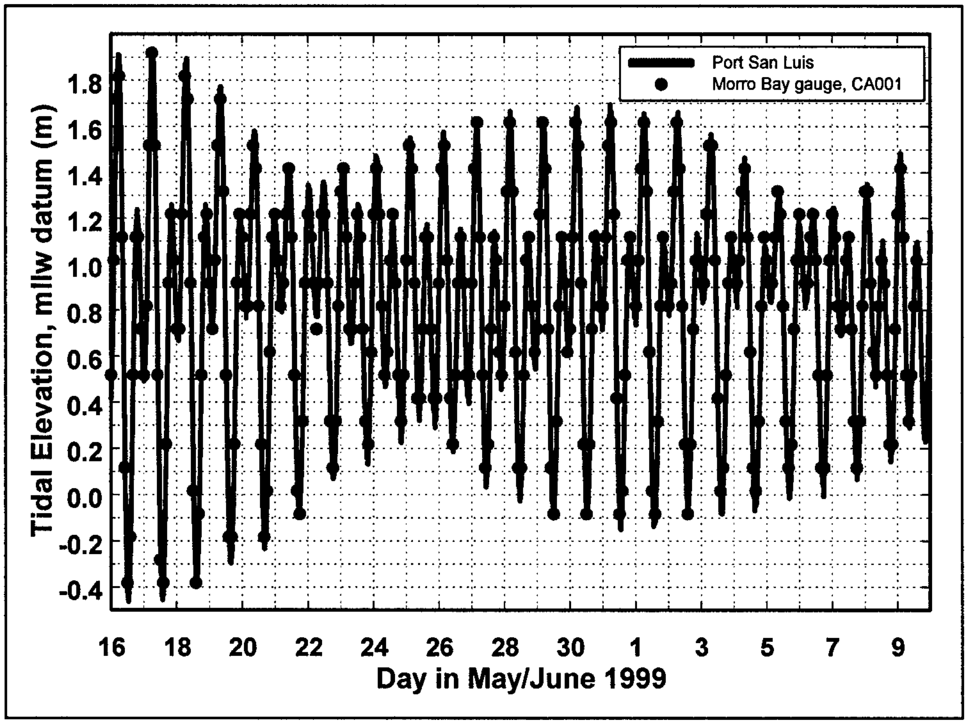

Figure 19. Measured time-history of tidal elevation, 16 May through 9 June 1999

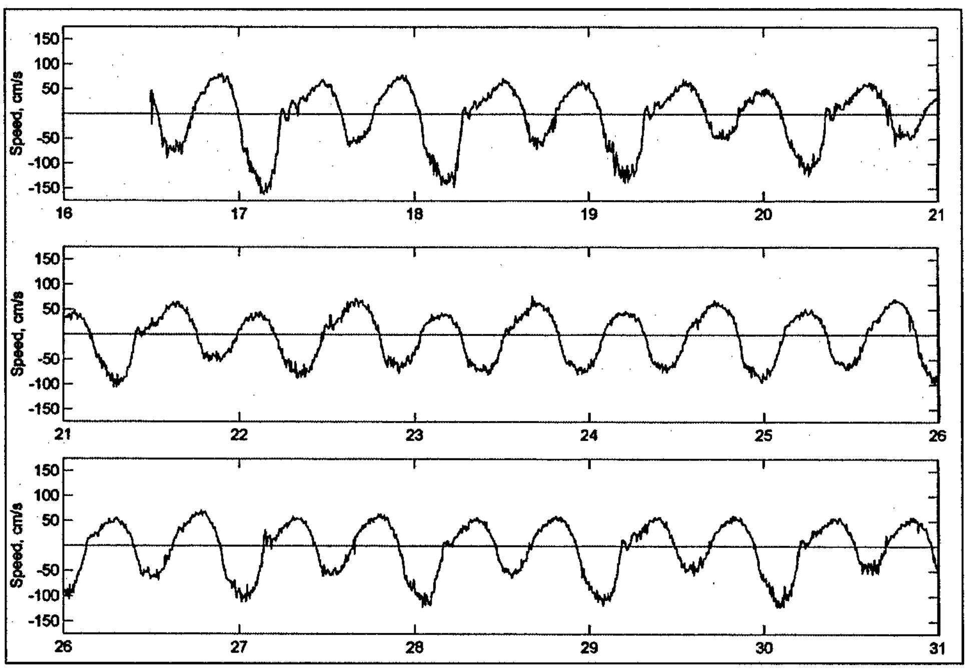

Figure 20. Ebb (-) and flood (+) near-surface currents for May 16 – 30, 1999

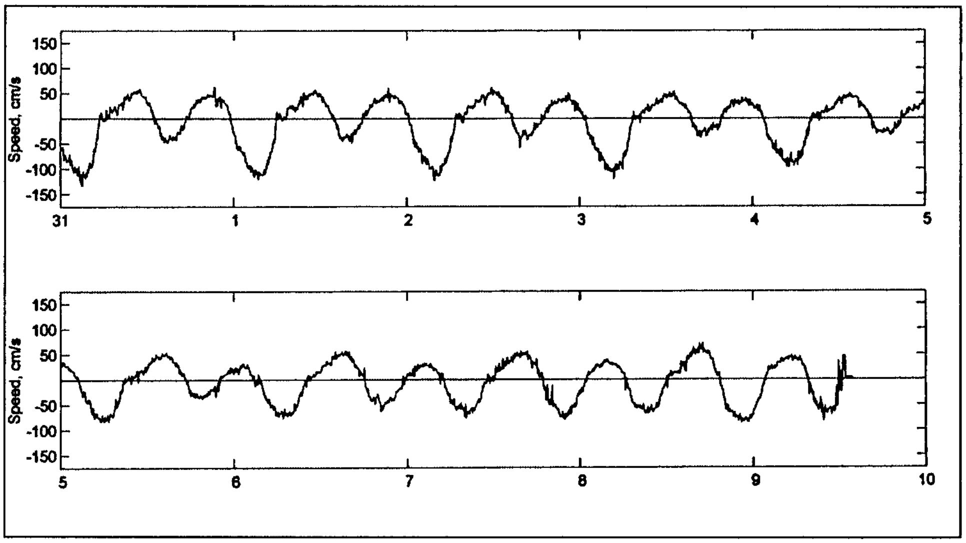

Figure 21. Ebb (-) and flood (+) near-surface currents for May 31 through June 9, 1999

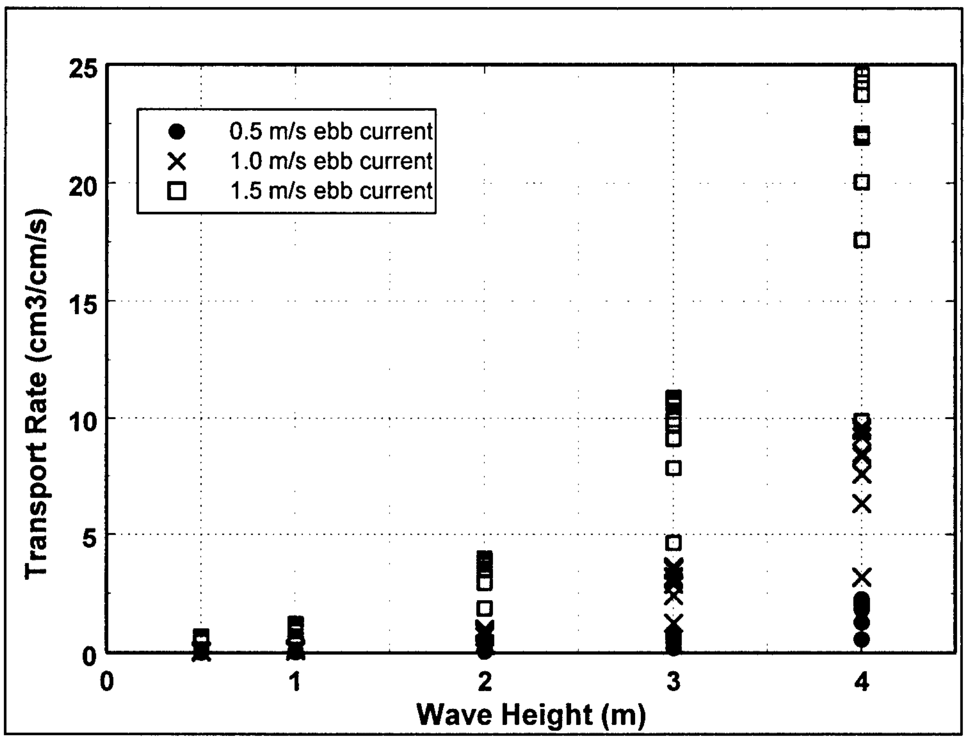

Figure 22. Model sediment transport rate due to waves and ebb currents, 9.1-m depth

12-16-sec periods and the lowest rates to 6-sec periods. Model results suggest that, within the range of conditions experienced at Morro Bay Harbor entrance, strong currents greatly amplify sediment transport rate during high wave conditions. For the higher wave heights considered, transport rate is a factor of

10 higher during 1.5 m/s (4.9 fps) currents than during 0.5 m/s (1.6 fps) currents. The model gave similar results for flood currents.

Bathymetric data results

Bathymetric data collected since initial dredging of the modified entrance and transition area at Morro Bay Harbor provide a valuable record of shoaling rates and patterns. Bottom changes over survey intervals, generally those unaffected by dredging in entrance and transition areas, are presented as gain-loss contour plots in Appendix C. The plots represent time intervals ranging from 1-15 months. Wave conditions characterizing each survey interval vary from mild summer waves to intense storms to the full range of seasons. During survey intervals encompassing winter months, gain-loss plots show shoaling in the modified entrance, transition, sand trap, and main channel areas. During survey intervals covering predominantly summer months, bathymetric change is slight in most areas. Areas consistently prone to shoaling include the entrance and transition areas and the east side of the main channel along the outer part of the groin. The plots show evidence of severe storms blanketing the modified entrance with 1-2 m (3.3-6.6 ft) of sediment from the south and scouring around the north breakwater head. Severe storms also appear to result in a sediment deposition of 2 m (6.6 ft) or more in the mid and upper part of the main channel. Patterns over the longer survey intervals suggest that shoals built during stormy periods are gradually redistributed by waves and currents.

To quantify bottom changes, the dredged project area was divided into nine segments, as shown in Figure 23. For areas A through F, several measures of bottom change were used to capture different aspects of project response to dredging. Methods for calculating available materials in these areas are defined in Figure 24. The volume of available materials was calculated from each survey for each area (Table 3). Calculations were done with INROADS, software used to create, display, and modify 3-D surfaces for civil engineering applications. Bottom changes between successive surveys provide useful data for bathymetric change analysis.

The volume of available materials provides a useful, easily understood measure of the amount of sediment stored in the dredged project areas. This volume varies with time due to deposition, scour, and dredging. Variation in available material volume with time over the length of the monitoring project is summarized in Figure 25. Values shown for the modified entrance are the sum of available materials for areas A, B, C, and D at -12.2 m (-40 ft). Values for the transition are the sum of available materials for areas E and F at -12.2 m (-40 ft) to -4.9 m (-16 ft). Sand trap values are for Area G at -7.6 m (-25 ft) and main channel values are the sum for areas H and I at -4.9 m (-16 ft).

The volume of available materials is greater for the modified entrance than for other areas during most of the 6-year time period. The second largest volume is generally in the sand trap. The volume stored in the modified entrance also varies more dramatically with time than for other areas. Variations represent increases due to natural shoaling punctuated by sharp decreases due to periodic dredging. The volume of materials in the main channel also shows cycles of

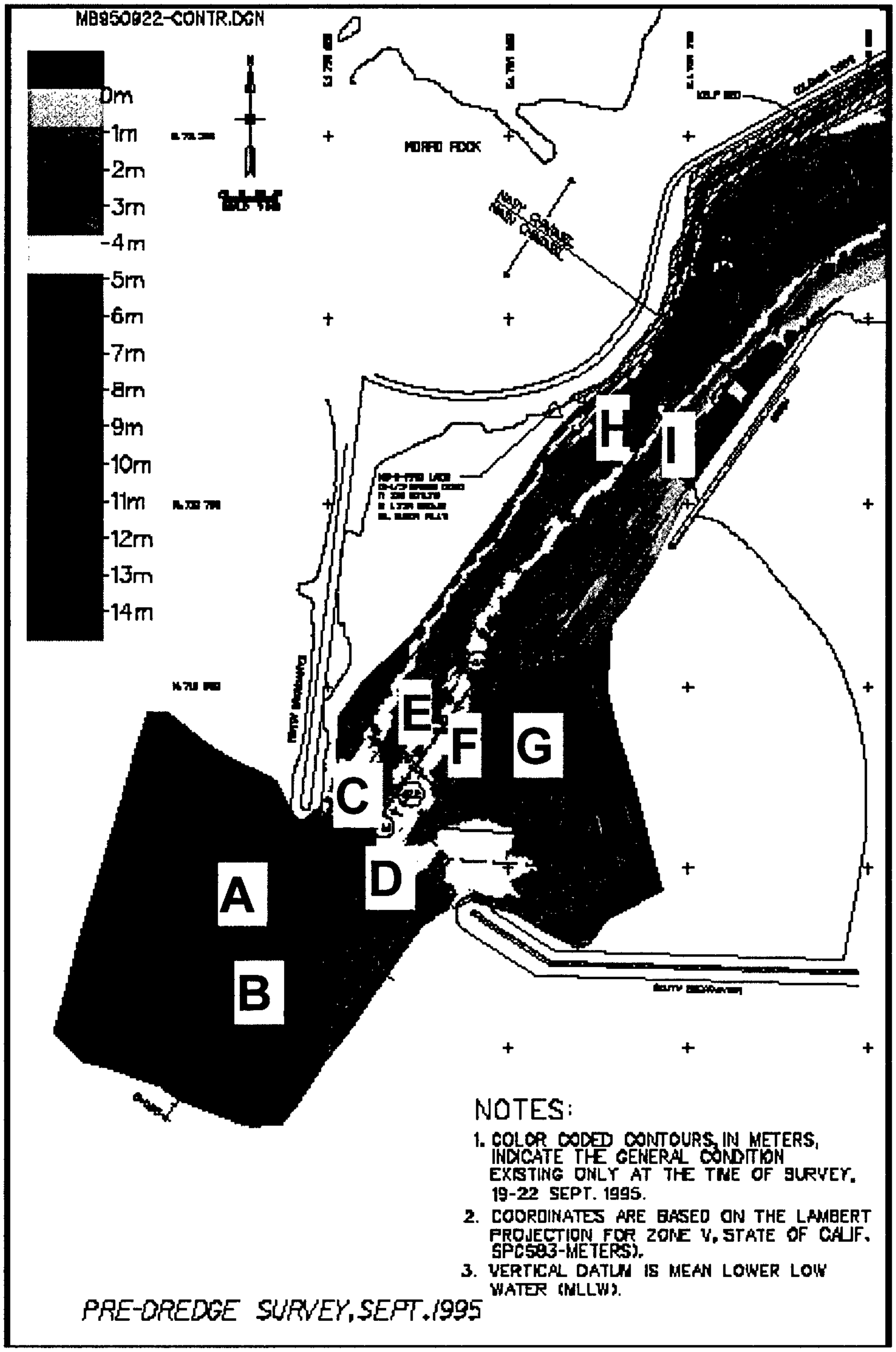

Figure 23. Definition of prototype areas for quantifying shoaling rate

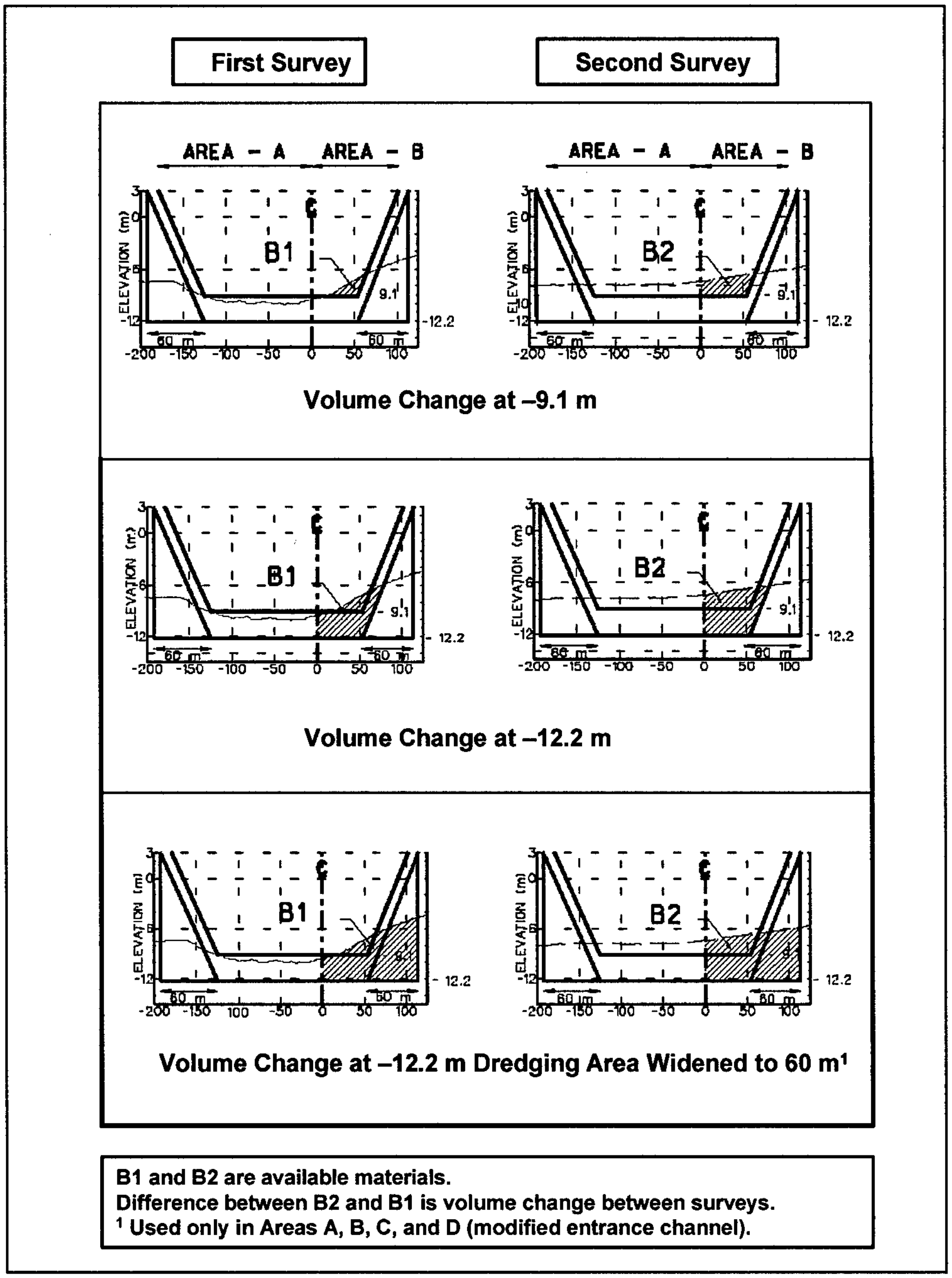

Figure 24. Definition sketch of methods for calculating available materials and volume changes

Table 3
Available Materials Calculated from Survey Data (cu m)

Area	Survey Date											
	09/22/95	12/10/95	12/26/95	04/04/96	07/11/97	09/24/97	10/09/97	01/06/98	01/29/98	02/27/98	04/21/98	07/29/98
A at –9.1m	37,140	0	316	609	4,750	4,581	2,557	2,777	8,050	26,647	4,164	5,362
A at –12.2 m	117,343	5,075	26,449	41,768	72,477	73,805	60,142	65,537	81,924	104,665	51,881	58,818
A at –12.2 m widened to 60 m	198,980	72,636	84,706	109,333	138,956	140,691	120,714	128,423	153,521	179,816	120,206	134,739
B at –9.1m	38,480	0	201	128	2,600	3,882	101	1,117	4,403	42,666	762	1,066
B at –12.2 m	129,353	4,632	38,504	45,009	63,064	67,583	53,396	61,215	78,011	136,750	42,108	50,709
B at –12.2 m widened to 60 m	242,132	88,436	124,184	123,892	137,650	143,083	120,710	128,708	162,682	264,753	145,886	160,788
C at –9.1m	18,490	1,012	677	596	4,486	6,694	4,291	9,773	11,400	10,646	3,999	3,827
C at –12.2 m	44,241	12,897	13,060	15,219	26,547	30,325	27,927	35,620	37,594	30,955	19,244	19,475
C at –12.2 m widened to 60 m	92,229	61,939	45,000	51,910	76,469	81,298	78,690	84,350	90,625	70,801	57,356	50,963
D at –9.1m	46,075	3,751	2,351	4,404	13,826	17,560	8,333	20,504	23,463	29,111	7,758	6,900
D at –12.2 m	87,137	23,613	28,298	36,675	51,572	56,281	45,391	58,282	61,984	69,421	35,504	35,691
D at –12.2 m widened to 60 m	127,790	63,542	64,248	71,519	84,721	90,070	78,036	91,740	97,179	109,167	72,785	73,242
E at –9.1 m to –4.9 m	16,825	1,403	1,547	3,640	6,043	7,199	3,461	8,509	9,725	18,588	6,724	8,777
E at –12.2 m to –4.9 m	19,203	2,966	3,146	5,465	7,701	8,922	5,087	10,517	11,801	20,640	8,929	11,041
F at –9.1 m to –4.9 m	29,176	23,642	22,156	5,217	10,016	10,303	6,599	15,486	15,642	18,038	3,604	3,137
F at –12.2 m to –4.9 m	30,335	24,640	23,149	6,376	11,175	11,461	7,758	16,644	16,801	19,197	4,316	3,857
G at –7.6 m	171,927	No data	No data	41,226	44,030	No data	48,198	78,801	77,829	109,738	126,916	125,397
H at –4.9 m	13,831	No data	No data	1,234	0	No data	0	5,018	4,220	39,278	51	176
I at –4.9 m	99,750	No data	No data	7,051	28,810	No data	26,539	63,184	76,321	103,904	22,698	25,495

(Continued)

Table 3 (Concluded)

Area	Survey Date								
	08/20/98	04/14/99	07/14/99	09/29/99	05/11/00	08/31/00	06/21/01	08/02/01	10/03/01
A at –9.1m	1,173	4,768	3,884	3,755	10,820	3,781	7,036	7,563	7,524
A at –12.2 m	33,324	74,467	64,894	65,438	86,044	65,825	81,002	81,600	81,218
A at –12.2 m widened to 60 m	108,433	145,832	135,159	135,751	154,477	131,057	150,116	147,857	150,655
B at –9.1m	1,046	1,585	521	566	12,217	566	9,450	10,450	10,009
B at –12.2 m	49,373	72,279	66,206	68,871	93,500	68,881	91,346	90,478	90,870
B at –12.2 m widened to 60 m	162,105	160,785	152,376	157,305	188,205	157,317	182,808	184,565	183,846
C at –9.1m	4,136	9,930	9,445	9,218	18,688	9,233	16,453	15,382	15,294
C at –12.2 m	18,899	34,847	35,143	34,995	45,061	35,043	43,125	41,425	41,427
C at –12.2 m widened to 60 m	54,115	77,272	80,457	81,050	79,471	70,982	88,996	83,137	83,177
D at –9.1m	3,764	12,106	6,008	6,271	26,213	6,269	28,879	27,320	27,856
D at –12.2 m	25,567	50,343	42,677	43,225	65,259	43,218	66,306	64,936	67,108
D at –12.2 m widened to 60 m	63,856	87,043	78,827	79,510	101,972	79,412	103,498	102,894	103,208
E at –9.1 m to –4.9 m	7,280	13,167	13,226	12,537	19,058	12,534	13,876	12,950	13,437
E at –12.2 m to –4.9 m	9,564	15,862	16,071	15,342	21,660	15,298	16,249	15,216	15,769
F at –9.1 m to –4.9 m	1,721	5,169	2,395	2,667	10,085	2,665	11,764	9,435	10,040
F at –12.2 m to –4.9 m	2,315	6,328	3,341	3,654	11,244	3,652	12,923	10,594	11,199
G at –7.6 m	124,978	123,349	122,895	124,863	124,565	124,871	131,618	132,721	132,518
H at –4.9 m	323	1,666	9	0	8,593	0	10,151	78	93
I at –4.9 m	17,214	67,896	29,142	37,039	86,504	36,028	83,666	21,622	25,636

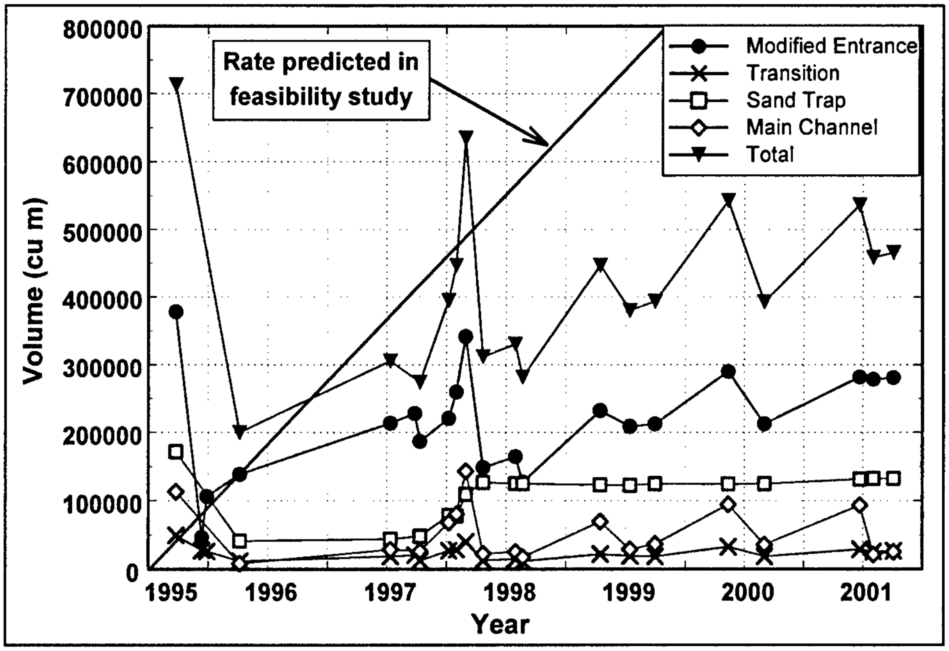

Figure 25. Summary of available materials versus time, calculated from survey data

shoal buildup and dredged removal. Such cycles are not evident in the sand trap, because most dredging episodes did not address that area. The sand trap filled and remained filled during most of the monitoring period.

As discussed earlier, storms can cause major shoaling, especially in the entrance. Storm-induced shoals are then gradually redistributed over time throughout the project area. Thus, the total volume of available materials in the dredged project area is probably a more accurate measure of shoaling induced by coastal waves and currents. Total volume is also shown in Figure 25. Total volume during the entire monitoring period is less than the preproject volume of 713,000 cu m in September 1995. Total volume reached its maximum value, around 630,000 cu m, during the winter 1998 storm season. Shoaling rate predicted by USAED, Los Angeles (1991) in the project feasibility study, 183,500 cu m/year (240,000 cu yd/year), is illustrated by a sloped line superimposed on the volume time-histories. The slope of this line is remarkably similar to slopes of total volume accumulation between dredging cycles during 1998-2001.

Survey intervals used in bathymetric change analysis were numbered sequentially for convenient reference (Table 4). The number of months and seasons characterizing each survey interval are also given.

Differences in available quantities between successive surveys were calculated to give volume changes and shoaling rates over each survey interval (Tables 5 and 6). These results relate to the gain-loss plots in Appendix C, but they provide quantitative data for the nine area segments. Shoaling rate data

Table 4
Survey Intervals for Bathymetric Change Analysis

Survey Interval Number	Survey Interval Dates	Number of Months	Seasons
1	26 Dec. 95 – 4 Apr. 96	3.3	Winter
2	4 Apr. 96 – 11 July 97	15.4	All seasons
3	11 July 97 – 24 Sept. 97	2.5	Summer
4	9 Oct. 97 – 6 Jan. 98	3.0	Fall
5	6 Jan. 98 – 29 Jan. 98	0.8	Winter
6	29 Jan. 98 – 27 Feb. 98	1.0	Winter
7	21 Apr. 98 – 29 July 98	3.3	Spring/summer
8	29 July 98 - 20 Aug. 98	0.7	Summer
9	20 Aug. 98 – 14 Apr. 99	7.9	All seasons
10	14 Apr. 99 – 14 July 99	3.0	Spring/summer
11	14 July 99 – 29 Sept. 99	2.6	Summer
12	29 Sept. 99 – 11 May 00	7.5	Fall/winter/spring
13	11 May 00 – 31 Aug. 00	3.7	Spring/summer
14	31 Aug. 00 – 21 June 01	9.8	All seasons
15	2 Aug. 01 – 3 Oct. 01	2.0	Summer

indicate that shoaling preferentially affects the south portion of the channel in the transition area and often in the modified entrance, as well.

Shoaling rates calculated from survey data are influenced by seasons represented in the survey interval. They may also be influenced by interval length, since shoaling rates may tend to be elevated immediately after dredging as material is more effectively trapped in deep excavation areas. The importance of these influences on the Morro Bay Harbor data is shown in Figure 26. Plotted shoaling rate is the total of all project areas considered, with volume definitions the same as those used for Figure 25. Shoaling rate clearly diminishes with increasing survey interval length. Seasonal effects are dramatic. Survey interval No. 6, an intense winter storm interval, produced shoaling rates an order of magnitude greater than for most other intervals. Survey interval No. 11, a summer interval, produced the lowest shoaling rate, despite its being one of the shortest survey intervals immediately following entrance dredging. The longest survey interval (No. 2) produced the lowest shoaling rate among non-summer intervals. This rate is even lower than the Los Angeles District-predicted rate. However, the 15.4 months encompassed by survey interval No. 2 include a disproportionate number of summer months and that can be expected to make the shoaling rate unrepresentatively low. The average of all survey intervals shown, taking into account interval length, is 19,300 cu m/month. The rate predicted in the feasibility study (USAED, Los Angeles, 1991) is 15,300 cu m/month.

Incident wave climate information can be helpful in analyzing bathymetric changes determined from survey data. Although linkages between incident waves and sediment transport and trapping around Morro Bay Harbor entrance are complex and poorly understood, incident waves clearly provide a major forcing. Incident wave information at the location of Gauge CA002 in 14.3-m (47-ft) depth was derived from offshore gauges, as discussed earlier, to cover all

Table 5
Volume Changes Calculated from Survey Data (cu m)

Area	Survey Interval Number											
	1	2	3	4	5	6	7	8	9	10	11	12
A at –9.1m	293	4,141	-169	220	Dredging	18,597	1,198	Dredging	3,595	Dredging	-129	7,065
A at –12.2 m	15,319	30,709	1,328	5,395	Dredging	22,741	6,937	Dredging	41,143	Dredging	544	20,606
A at –12.2 m widened to 60 m	24,627	29,623	1,735	7,709	Dredging	26,295	14,533	Dredging	37,399	Dredging	592	18,726
B at –9.1m	-73	2,472	1,282	1,016	Dredging	38,263	304	Dredging	539	Dredging	45	11,651
B at –12.2 m	6,505	18,055	4,519	7,819	Dredging	58,739	8,601	Dredging	22,906	Dredging	2,665	24,629
B at –12.2 m widened to 60 m	-292	13,758	5,433	7,998	Dredging	102,071	14,902	Dredging	-1,320	Dredging	4,929	30,900
C at –9.1m	-81	3,890	2,208	5,482	Dredging	-754	-172	Dredging	5,794	Dredging	-227	9,470
C at –12.2 m	2,159	11,328	3,778	7,693	Dredging	-6,639	231	Dredging	15,948	Dredging	-148	10,066
C at –12.2 m widened to 60 m	6,910	24,559	4,829	5,660	Dredging	-19,824	-6,393	Dredging	23,157	Dredging	593	-1,579
D at –9.1m	2,053	9,422	3,734	12,171	Dredging	5,648	-858	Dredging	8,342	Dredging	263	19,942
D at –12.2 m	8,377	14,897	4,709	12,891	Dredging	7,437	187	Dredging	24,776	Dredging	548	22,034
D at –12.2 m widened to 60 m	7,271	13,202	5,349	13,704	Dredging	11,988	457	Dredging	23,187	Dredging	683	22,462
E at –9.1 m to –4.9 m	Dredging	2,403	1,156	5,048	Dredging	8,863	2,053	Dredging	5,887	Dredging	-689	6,521
E at –12.2 m to –4.9 m	Dredging	2,236	1,221	5,430	Dredging	8,839	2,112	Dredging	6,298	Dredging	-729	6,318
F at –9.1 m to –4.9 m	Dredging	4,799	287	8,887	Dredging	2,396	-467	Dredging	3,448	Dredging	272	7,418
F at –12.2 m to –4.9 m	Dredging	4,799	286	8,886	Dredging	2,396	-459	Dredging	4,013	Dredging	313	7,590
G at –7.6 m	No data	2,804	No data	30,603	-	31,909	-	-419	-1,629	-454	1,968	-298
H at –4.9 m	No data	-1,234	No data	5,018	-798	35,058	125	147	1,343	Dredging	-9	8,593
I at –4.9 m	No data	21,759	No data	36,645	13,137	27,583	2,797	Dredging	50,682	Dredging	7,897	49,465

(Continued)

Table 5 (Concluded)

Area	Survey Interval Number		
	13	14	15
A at –9.1m	Dredging	3,255	-39
A at –12.2 m	Dredging	15,177	-382
A at –12.2 m widened to 60 m	Dredging	19,059	2,798
B at –9.1m	Dredging	8,884	-441
B at –12.2 m	Dredging	22,465	392
B at –12.2 m widened to 60 m	Dredging	25,491	-719
C at –9.1m	Dredging	7,220	-88
C at –12.2 m	Dredging	8,082	2
C at –12.2 m widened to 60 m	Dredging	18,014	40
D at –9.1m	Dredging	22,610	536
D at –12.2 m	Dredging	23,088	2,172
D at –12.2 m widened to 60 m	Dredging	24,086	314
E at –9.1 m to –4.9 m	Dredging	1,342	487
E at –12.2 m to –4.9 m	Dredging	951	553
F at –9.1 m to –4.9 m	Dredging	9,099	605
F at –12.2 m to –4.9 m	Dredging	9,271	605
G at –7.6 m	306	6,747	-203
H at –4.9 m	Dredging	10,151	15
I at –4.9 m	Dredging	47,638	4,014

Table 6
Shoaling Rates Calculated from Survey Data (cu m/month)

Area	Survey Interval Number														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
A at –9.1m	89	272	-70	76	-	19,997	363	-	461	-	-52	955	-	336	-20
A at –12.2 m	4,642	2,020	553	1,860	-	24,453	2,102	-	5,275	-	218	2,785	-	1,565	-191
A at –12.2 m widened to 60 m	7,463	1,949	723	2,658	-	28,274	4,404	-	4,795	-	237	2,531	-	1,965	1,399
B at –9.1m	-22	163	534	350	-	41,143	92	-	69	-	18	1,574	-	916	-221
B at –12.2 m	1,971	1,188	1,883	2,696	-	63,160	2,606	-	2,937	-	1,066	3,328	-	2,316	196
B at –12.2 m widened to 60 m	-88	905	2,264	2,758	-	109,754	4,516	-	-169	-	1,972	4,176	-	2,628	-360
C at –9.1m	-25	256	920	1,890	-	-811	-52	-	743	-	-91	1,280	-	744	-44
C at –12.2 m	654	745	1,574	2,653	-	-7,139	70	-	2,045	-	-59	1,360	-	833	1
C at –12.2 m widened to 60 m	2,094	1,616	2,012	1,952	-	-21,316	-1,937	-	2,969	-	237	-213	-	1,857	20
D at –9.1m	622	620	1,556	4,197	-	6,073	-260	-	1,069	-	105	2,695	-	2,331	268
D at –12.2 m	2,538	980	1,962	4,445	-	7,997	57	-	3,176	-	219	2,978	-	2,380	1,086
D at –12.2 m widened to 60 m	2,203	869	2,229	4,726	-	12,890	138	-	2,973	-	273	3,035	-	2,483	157
E at –9.1 m to –4.9 m	-	158	482	1,741	-	9,530	622	-	755	-	-276	881	-	138	244
E at –12.2 m to –4.9 m	-	147	509	1,872	-	9,504	640	-	807	-	-292	854	-	98	277
F at –9.1 m to –4.9 m	-	316	120	3,064	-	2,576	-142	-	442	-	109	1,002	-	938	303
F at –12.2 m to –4.9 m	-	316	119	3,064	-	2,576	-139	-	514	-	125	1,026	-	956	303
G at –7.6 m	-	184	-	10,553	-	34,311	-	-	-209	-	787	-40	-	696	-102
H at –4.9 m	-	-81	-	1,730	-1,036	37,697	38	-	172	-	-4	1,161	-	1,046	8
I at –4.9 m	-	1,432	-	12,636	17,061	29,659	848	-	6,498	-	3,159	6,684	-	4,911	2,007

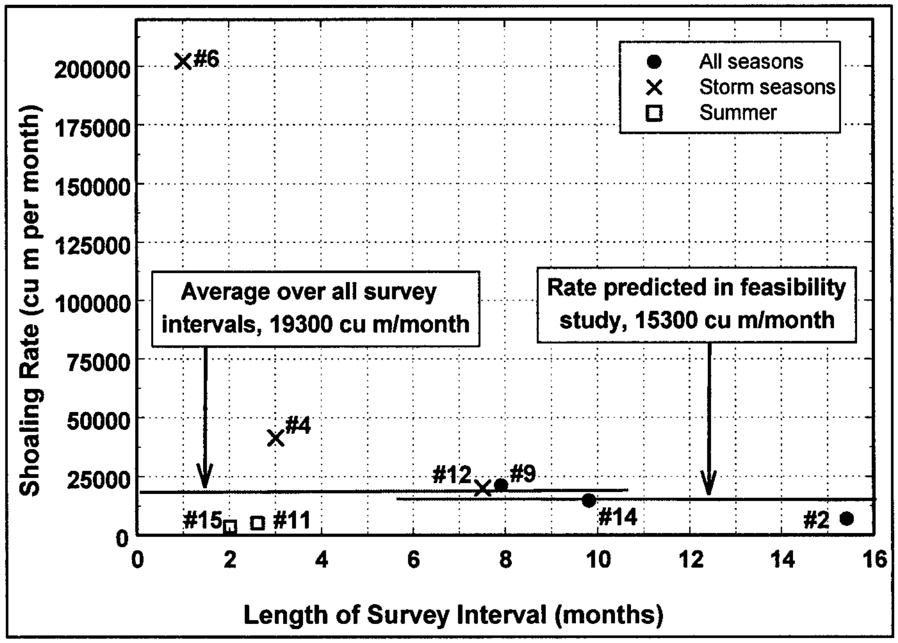

Figure 26. Summary of total shoaling rate from survey data versus length of survey interval; survey interval numbers included

survey intervals used for bathymetric change analysis. Time-histories of incident wave significant height, period, and direction were further processed to estimate nearshore breaking wave parameters and potential wave-driven longshore sediment transport. Straight, parallel bottom contours were assumed between 14.3-m (47-ft) depth and the point of wave breaking.

A standard equation was used for the calculations, as follows:

$$Q = K\, H_{bs}^{5/2} \sin\left(2\alpha_b\right)$$

where

Q = potential longshore transport rate

K = constant

H_{bs} = significant wave height at breaking

α_b = breaking wave angle relative to bottom contours

When H_{bs} is in meters and Q in m^3/day, the traditional value of K is 5,100. Several recent studies applying the equation as in this study have found that this value of K overestimates longshore transport rates relative to field experience and a value of K = 1,987 is more appropriate (e.g., Cialone and Thompson 2000). The lower value of K was also used in this study.

Calculated potential longshore transport rate time-histories were summed to give potential longshore transport volumes over each survey interval. Calculated volumes include northward transport, southward transport, net transport (difference between northward and southward transport), and gross transport (sum of northward and southward transport). Calculated longshore transport volumes over the survey intervals and corresponding monthly rates are given in Table 7. Southward transport is considered negative, following normal longshore transport sign convention. Gross transport, Q_g, is taken as the volume most relevant to sediment entrapment in the dredged project area.

Potential gross longshore transport volumes computed from wave time-histories are given in Table 8 along with summarized volume changes from survey data. The relationship between them is varied (Figure 27). Although volume changes are shown for project segments (e.g., modified entrance, transition area, etc.), the total volume change is considered most meaningful. For the two shortest non-summer survey intervals (No. 4 and No. 6), the total volume change from survey data equals or exceeds Q_g. This suggests that 100 percent of the gross longshore transport was captured in the newly-dredged project during these two stormy survey intervals. For longer survey intervals, total volume change is less than one-third of the potential gross longshore transport volume. The data suggest that, over time intervals on the order of 1 year, around 15-30 percent of the gross longshore transport may be retained in the dredged project.

Potential longshore transport volumes are also helpful for giving a perspective on the anomalous volume changes calculated during February 1998 (survey interval No. 6). Potential Q_g volume for this 1-month interval is comparable to that for survey interval No. 4, a 3-month interval during fall and early winter of the same year. Intensified storm activity indicated during the winter of 1997-1998 may attributed to the presence of El Niño this year. Potential Q_g volumes for longer survey intervals are considerably larger than for the winter of 1997-1998, suggesting that the net effect of this winter is reasonably consistent with other years.

Photogrammetric survey results

Prior to the photogrammetric survey work at the Morro Bay south breakwater, limited ground surveys were conducted. Targets established on the breakwater are shown in Appendix D for the June 1998 and July 2000 surveys. Thirty targets were initially established for the 1998 survey, which were used for control for the 1998 photogrammetric flight. For the 2000 survey, not all targets were recovered due to excessive bird droppings on the structure and the rush to obtain data before commencement of the rehabilitation work. Twenty targets were used for control for the 2000 photogrammetric flight. Of these, three were new re-established targets, and 17 were recovered and corresponded to those of the 1998 survey. Positions and elevations of the targets are presented in Table 9 for the two surveys, as well as differences obtained in horizontal and vertical directions for the 17 recovered targets used in both surveys. Maximum movement occurred for target 24, which was located on the sea slope of the breakwater at approximately sta 13+06. The targeted stone moved greater than

Table 7
Potential Longshore Transport Calculated from Wave Data

Survey Interval Number	Wave Data Source	Longshore Transport Volume During Survey Interval cu m				Longshore Transport Rate During Survey Interval cu m/month			
		Southward	Northward	Net	Gross	Southward	Northward	Net	Gross
1	Monterey buoy	-150,030	26,220	-123,820	176,250	-45,010	7,870	-37,150	52,880
2	Monterey buoy	-704,860	61,620	-643,240	766,480	-45,570	3,980	-41,590	49,560
3	Monterey buoy	-55,870	10,050	-45,820	65,930	-22,060	3,970	-18,090	26,020
4	Harvest slope array	-106,510	17,450	-89,060	123,950	-35,500	5,820	-29,690	41,320
6	Harvest slope array	-85,750	35,090	-50,660	120,830	-85,750	35,090	-50,660	120,830
7	Harvest buoy	-120,260	8,370	-111,900	128,630	-36,080	2,510	-33,570	38,590
9	Harvest buoy Monterey buoy	-529,510 -573,520	5,250 10,690	-524,260 -562,830	534,760 584,220	-66,750 -72,290	660 1,350	-66,080 -70,940	67,410 73,640
11	Harvest buoy Monterey buoy	-68,860 -45,690	4,670 4,910	-64,190 -40,780	73,520 50,600	-26,480 -22,850	1,790 2,450	-24,690 -20,390	28,280 25,300
12	Harvest buoy Monterey buoy	-433,300 -384,220	18,620 27,480	-414,680 -356,740	451,930 411,710	-57,770 -51,230	2,480 3,660	-55,290 -47,570	60,260 54,890
14	Harvest buoy Monterey buoy	-543,870 -552,560	14,500 19290	-529,370 -533,270	558,360 571,850	-55,310 -56,190	1,470 1,960	-53,830 -54,230	56,780 58,150

Table 8
Summary of Potential Longshore Transport and Volume Changes from Survey Data

Survey Interval Number	Potential Longshore Transport Volume During Survey Interval		Volume Change from Survey Data (cu m)				
	Wave Data Source	Q_g, cu m	Modified Entrance[1]	Transition[2]	Sand Trap[3]	Main Channel[4]	Total
1	Monterey buoy	176,250	32,360	Dredging	No data	No data	-
2	Monterey buoy	766,480	74,989	7,035	2,804	20,525	105,353
3	Monterey buoy	65,930	14,334	1,507	No data	No data	-
4	Harvest slope array	123,950	33,798	14,316	30,603	41,663	120,380
5			Dredging				
6	Harvest slope array	120,830	82,278	11,235	31,909	62,641	188,063
7	Harvest buoy	128,630	15,956	1,653	-	2,922	-
8			Dredging				
9	Harvest buoy Monterey buoy	534,760 584,220	104,773	10,311	-1,629	52,025	165,480
10			Dredging				
11	Harvest buoy Monterey buoy (partial)	73,520 50,600	3,609	-416	1,968	7,888	13,049
12	Harvest buoy Monterey buoy	451,930 411,710	77,335	13,908	-298	58,058	149,003
13			Dredging				
14	Harvest buoy Monterey buoy	558,360 571,850	68,812	10,222	6,747	57,789	143,570
15	-	-	2,184	1,158	-203	4,029	7,168

[1] Total of Areas A, B, C, and D at –12.2 m
[2] Total of Areas E and F at –12.2 m to –4.9 m
[3] Area G at –7.6 m
[4] Total of Areas H and I at –4.9 m

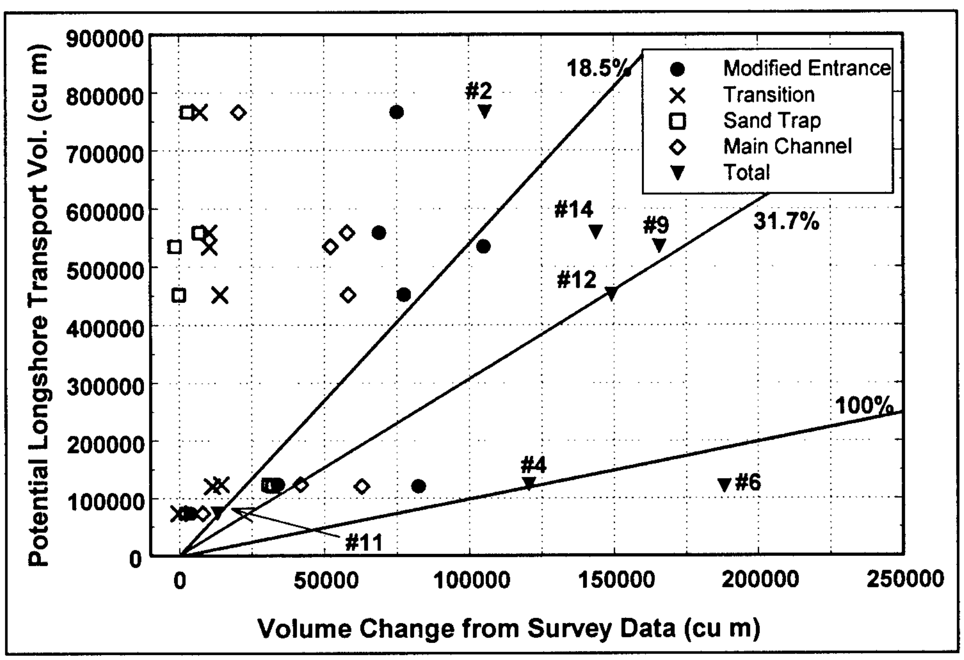

Figure 27. Potential gross longshore transport volume versus volume change from survey data; survey interval numbers included; lines indicate representative percentages of potential longshore transport volume

Table 9
Data Obtained for Targets and Differences in Positions for 1998 and 2000 Photogrammetric Surveys

	1998 Survey			2000 Survey			Difference in Positions, m (ft)		
Target	Easting	Northing	El, m (ft)	Easting	Northing	El, m (ft)	Easting	Northing	El, m (ft)
1	5706871.50	2329985.54	3.88 (12.72)	5706871.61	2329985.50	3.86 (12.66)	0.036 (0.12)	0.012 (0.04)	-0.018 (-0.06)
2	5706856.73	2329997.93	3.53 (11.59)	---	---	---	---	---	---
3	5706873.46	2330005.19	3.30 (10.83)	5706873.55	2330005.33	3.28 (10.77)	0.027 (0.09)	0.043 (0.14)	-0.018 (-0.06)
4	5706876.54	2329994.70	3.58 (11.75)	5706876.51	2329994.83	3.57 (11.70)	0.009 (0.03)	0.040 (0.13)	-0.015 (-0.05)
5	5706937.45	2329960.66	4.69 (15.38)	5706937.45	2329960.66	4.69 (15.38)	0	0	0
6	5706944.02	2329981.29	2.69 (8.83)	---	---	---	---	---	---
606	---	---	---	5706941.02	2329980.27	2.45 (8.05)	---	---	---
7	5706931.78	2329943.96	3.21 (10.52)	---	---	---	---	---	---
707	---	---	---	5706947.79	2329935.42	3.51 (11.52)	---	---	---
8	5707014.14	2329907.82	3.84 (12.59)	5707014.14	2329907.86	3.84 (12.59)	0	0.012 (0.04)	0
9	5707014.98	2329926.43	1.91 (6.27)	5707015.01	2329926.44	1.92 (6.29)	0.009 (0.03)	0.003 (0.01)	0.006 (0.02)
10	5707013.59	2329903.34	4.05 (13.28)	---	---	---	---	---	---
500	---	---	---	5707013.60	2329903.41	4.05 (13.28)	---	---	---
11	5707018.23	2329887.88	3.62 (11.88)	5707018.19	2329887.99	3.61 (11.86)	0.012 (0.04)	0.034 (0.11)	-0.006 (-0.02)
12	5707088.31	2329862.62	4.57 (15.00)	5707088.32	2329862.59	4.58 (15.01)	0.003 (0.01)	0.009 (0.03)	0.003 (0.01)
13	5707090.80	2329879.09	1.69 (5.54)	5707090.75	2329879.14	1.70 (5.58)	0.015 (0.05)	0.015 (0.05)	0.012 (0.04)
14	5707079.17	2329848.92	3.57 (11.70)	5707079.15	2329848.92	3.56 (11.67)	0.006 (0.02)	0	-0.009 (-0.03)

(Continued)

Table 9 (Concluded)

Target	1998 Survey			2000 Survey			Difference in Positions, m (ft)		
	Easting	Northing	El, m (ft)	Easting	Northing	El, m (ft)	Easting	Northing	El, m (ft)
15	5707153.96	2329812.80	4.62 (15.16)	5707153.94	2329812.78	4.62 (15.16)	0.006 (0.02)	0.006 (0.02)	0
16	5707147.33	2329803.71	4.11 (13.48)	5707147.28	2329803.61	4.10 (13.45)	0.015 (0.05)	0.030 (0.10)	-0.009 (-0.03)
17	5707169.11	2329833.75	1.98 (6.49)	5707169.09	2329833.75	1.98 (6.51)	0.006 (0.02)	0	0.006 (0.02)
18	5707156.13	2329818.10	4.56 (14.95)	5707156.14	2329818.11	4.56 (14.97)	0.003 (0.01)	0.003 (0.01)	0.006 (0.02)
19	5707216.63	2329772.47	4.55 (14.93)	---	---	---	---	---	---
20	5707213.82	2329756.71	3.47 (11.37)	---	---	---	---	---	---
21	5707227.08	2329807.83	1.98 (6.48)	---	---	---	---	---	---
22	5707234.31	2329771.63	4.90 (16.06)	---	---	---	---	---	---
23	5707327.25	2329773.83	4.89 (16.03)	5707327.80	2329773.92	4.90 (16.07)	0.168 (0.55)	0.027 (0.09)	0.012 (0.04)
24	5707326.50	2329756.77	4.43 (14.52)	5707327.94	2329751.38	3.63 (11.91)	0.439 (1.44)	1.643 (5.39)	-0.796 (-2.61)
25	5707327.97	2329797.86	2.26 (7.42)	5707327.74	2329797.93	2.27 (7.45)	0.070 (0.23)	0.021 (0.07)	0.009 (0.03)
26	5706886.37	2329963.33	2.41 (7.91)	---	---	---	---	---	---
27	5707019.24	2329878.45	2.20 (7.23)	---	---	---	---	---	---
28	5707157.44	2329787.34	2.65 (8.69)	---	---	---	---	---	---
29	5707327.60	2329746.56	3.04 (9.98)	---	---	---	---	---	---
100	5707200.72	2329786.79	5.17 (16.96)	---	---	---	---	---	---

1.5 m (5 ft) horizontally and greater than 0.75 m (2.5 ft) vertically. The average movement of the other 16 targeted stones was 0.03 m (0.1 ft) in the horizontal direction and 0.009 m (0.03 ft) vertically.

Photographic stereo pairs secured for the outer south breakwater are shown in Figures 28-30. After orientation in the stereomodel to the monument and target data previously obtained, orthophotos were developed. Upon rectification, the accuracy of photogrammetric spot elevations were on the order of ±9 cm (±0.03 ft). Orthophotos developed for the July 2000 survey for the outer south breakwater are presented in Appendix E. In addition, contour maps of the breakwater as well as cross sections were developed from the DTM for the 1998 and 2000 surveys.

An examination of the breakwater topography for the 1998 and 2000 surveys indicated that the outer 91.4 m (300 ft) of the structure (sta 15+50 - 18+50) is below its design el of +4.9 m (+16 ft). Low areas in the breakwater are concentrated between stas 15+70 and 16+90 and stas 18+10 and 18+50. Elevations in these areas generally range from +4 to+4.6 m (+13 to +15 ft). Except for a few isolated cases, comparisons of the contours for the 2000 survey were almost identical to those of the 1998 survey. Topography of the breakwater in 2000 is shown in Appendix F.

During the photogrammetric analysis, the high points on 122 individual armor stones, scattered thoughout the monitored structure, was obtained and x, y, and z coordinates were noted in the 1998 survey. These points were not targeted, but selected from the stereomodel. The identical horizontal coordinate points were revisted during the 2000 survey and elevations obtained in an effort to determine if movement had occurred. The locations of the high points of the stones are shown in Appendix G. The coordinates of these points, elevations for the 1998 and 2000 surveys, and differences in elevations are presented in Table 10. Changes in elevations of only five of the 122 high points were greater than 0.3 m (1 ft). The average change in elevation of the remaining 117 high points was only 0.037 m (0.12 ft). The maximum elevation change was -1.314 m (-4.31 ft) at high point 51, located on the harbor side of the structure at sta 16+49. Also on the harbor side of the breakwater, high points 25 at sta 17+60, 45 at sta 16+73, and 79 at sta 15+23 lost 0.783, 0.53, and 0.338 m (2.57, 1.74, and 1.11 ft) in elevation, respectively. High point 80, situated on the sea side of the structure at sta 15+09, lost 0.756 (2.48 ft) in elevation.

Cross sections of the outer south breakwater developed from the 1998 and 2000 surveys are shown in Appendix H. Examination of these data reveal that most cross sections were similar for both surveys. Changes of 0.3 m (1 ft) or less occurred at most locations. The stone armor at sta 16+50 appears to have rearranged slightly.

In summary, the photogrammetric surveys of the Morro Bay Harbor south breakwater were very effective in accurately mapping the above-water portion of the structure and showing changes occurring from 1998 to 2000. The breakwater was in somewhat of a deteriorated state when the monitoring was initiated. However, except for a few isolated instances on the breakwater trunk, minimal

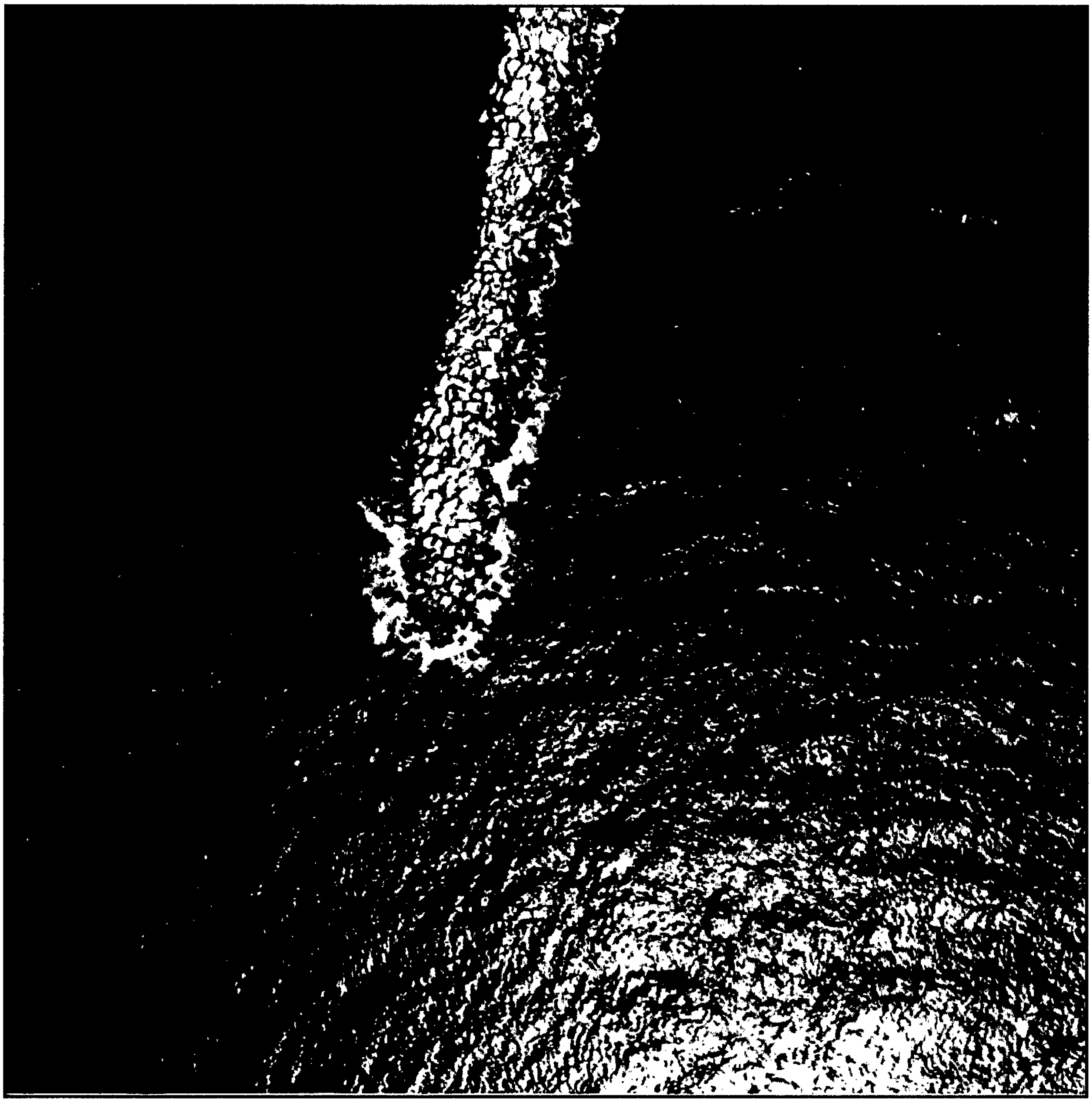

Figure 28. Stereo pair photo for south breakwater in 2000 (westernmost image)

changes occurred between 1998 and 2000. It was noted that very little change occurred at the head of the south breakwater. The physical model study conducted for Morro Bay Harbor indicated that the deepened entrance would result in slightly increased wave heights at the breakwater head. It was determined, however, at that point, that the increased wave heights would not exceed the design wave height for the structure, and therefore, should not result in breakwater damage. The monitoring effort, even though conducted for a limited time (2 years), appears to confirm this hypothesis.

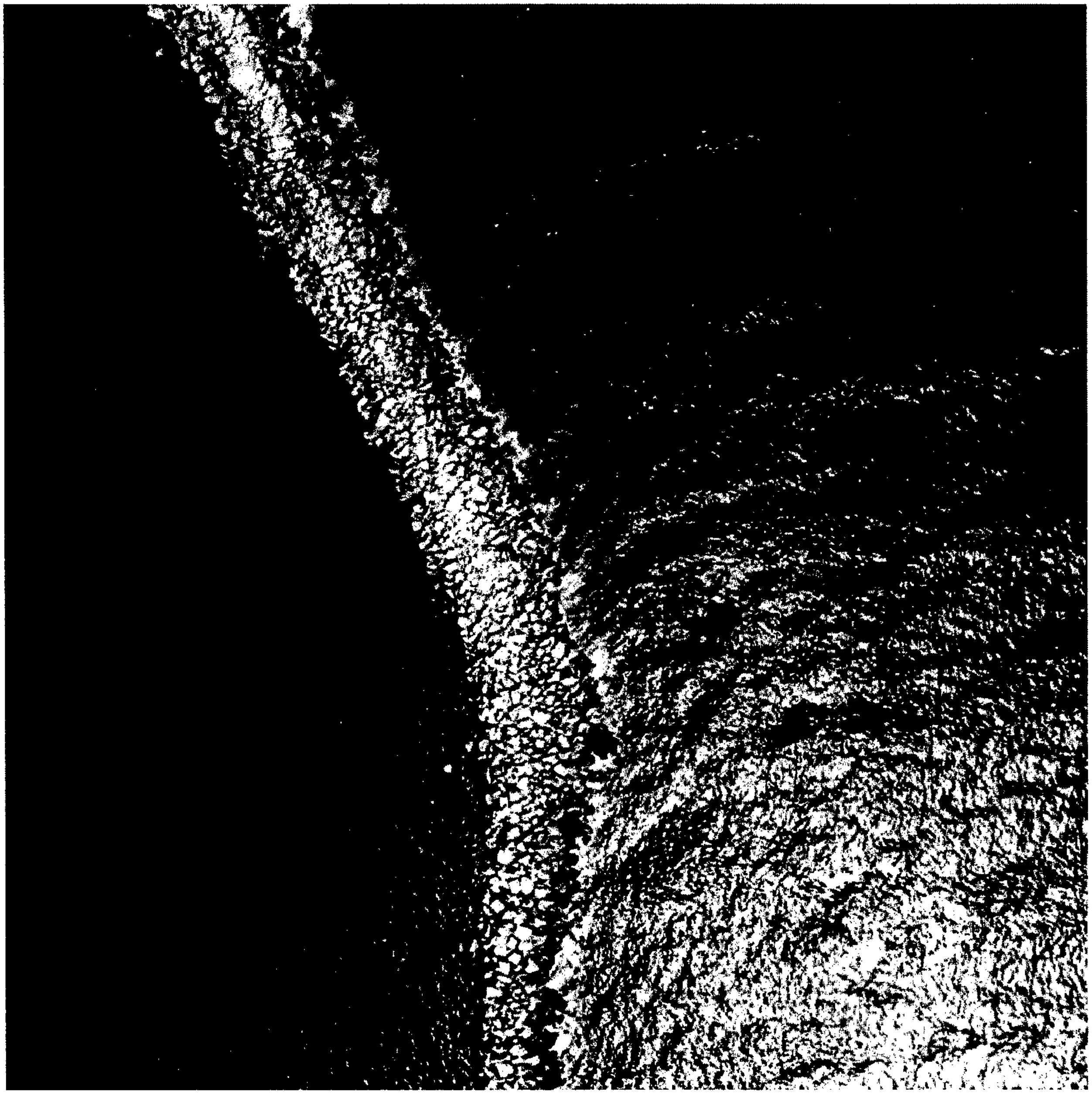

Figure 29. Stereo pair photo for south breakwater in 2000 (middle image)

Rehabilitation of the breakwater was completed during the fall of 2000. The Los Angeles District obtained additional photogrammetric data for the rehabilitated structure to serve as a base for determining breakwater changes in future years.

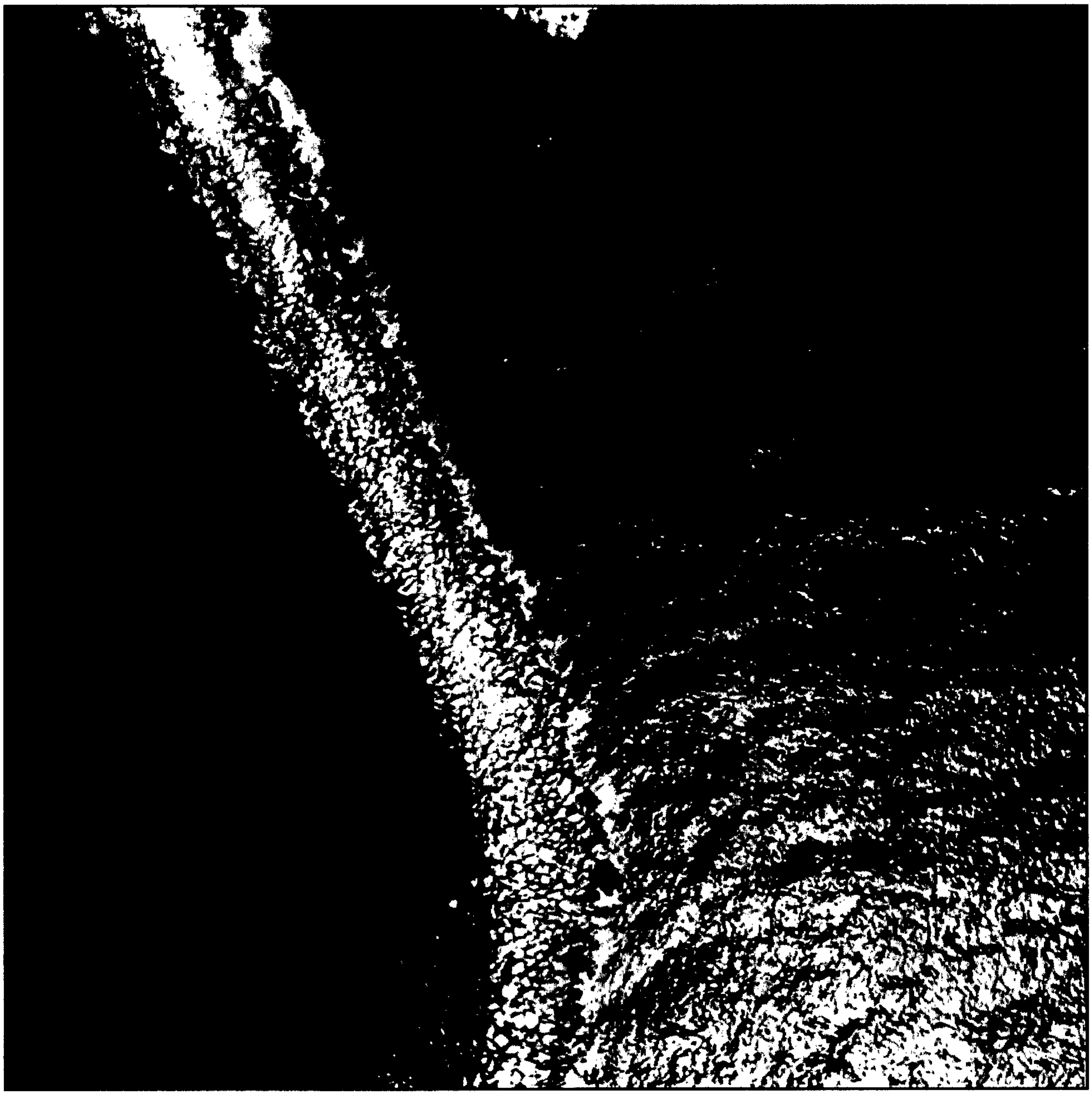

Figure 30. Stereo pair photo for south breakwater in 2000 (easternmost image)

Table 10
Data Obtained at High Points and Differences in Elevations for 1998 and 2000 Photogrammetric Surveys

High Point	Easting	Northing	1998 Survey El, m (ft)	2000 Survey El, m (ft)	Difference in El, m (ft)
1	5706844.07	2329998.95	2.43 (7.98)	2.58 (8.47)	0.149 (0.49)
2	5706847.23	2330010.39	2.76 (9.04)	2.68 (8.81)	-0.070 (-0.23)
3	5706850.55	2329992.29	3.01 (9.86)	3.05 (10.01)	0.046 (0.15)
4	5706857.01	2329985.10	3.15 (10.35)	3.18 (10.43)	0.024 (0.08)
5	5706857.43	2330015.50	2.33 (7.66)	2.37 (7.79)	0.040 (0.13)
6	5706866.74	2330002.34	3.97 (13.04)	4.02 (13.20)	0.049 (0.16)
7	5706868.66	2330011.82	3.39 (11.13)	3.25 (10.67)	-0.140 (-0.46)
8	5706869.14	2329982.17	3.04 (9.99)	3.10 (10.17)	0.055 (0.18)
9	5706876.46	2329974.12	2.95 (9.67)	2.94 (9.66)	-0.003 (-0.01)
10	5706879.30	2329994.75	3.47 (11.38)	3.47 (11.39)	0.003 (0.01)
11	5706881.77	2330008.63	1.9 (6.34)	1.98 (6.49)	0.046 (0.15)
12	5706885.01	2329968.98	3.75 (12.31)	3.84 (12.59)	0.085 (0.28)
13	5706885.10	2329988.06	4.66 (15.28)	4.65 (15.25)	-0.009 (-0.03)
14	5706894.68	2329997.60	2.55 (8.38)	2.64 (8.66)	0.085 (0.28)
15	5706900.85	2329963.02	2.65 (8.71)	2.65 (8.69)	-0.006 (-0.02)
16	5706902.99	2329983.15	5.07 (16.63)	5.12 (16.81)	0.055 (0.18)
17	5705903.29	2329954.17	3.20 (10.51)	3.70 (10.64)	0.040 (0.13)
18	5705909.90	2329968.70	4.62 (15.16)	4.68 (15.34)	0.055 (0.18)
19	5706912.76	2330008.36	2.07 (6.80)	2.10 (6.88)	0.024 (0.08)
20	5706913.50	2329995.04	3.20 (10.51)	3.28 (10.75)	0.073 (0.24)
21	5706916.59	2329957.25	3.14 (10.29)	3.19 (10.47)	0.055 (0.18)
22	5706919.70	2329972.45	4.66 (15.28)	4.66 (15.28)	0 (0)
23	5706925.90	2329946.38	2.65 (8.71)	2.65 (8.69)	-0.006 (-0.02)
24	5706930.35	2329951.74	4.13 (13.56)	4.12 (13.53)	-0.09 (-0.03)
25	5706934.82	2329975.06	3.55 (11.64)	2.76 (9.06)	-0.786 (-2.58)
26	5706935.10	2329987.54	2.13 (6.99)	2.23 (7.31)	0.098 (0.32)
27	5706941.19	2329938.97	3.33 (10.92)	3.32 (10.90)	-0.006 (-0.02)
28	5706944.67	2329959.99	4.83 (15.86)	4.86 (15.94)	0.024 (0.08)
29	5706951.38	2329937.67	3.87 (12.70)	3.92 (12.85)	0.046 (0.15)
30	5706951.76	2329925.28	2.65 (8.68)	2.63 (8.64)	-0.012 (-0.04)
31	5706954.62	2329937.27	4.38 (14.38)	4.50 (14.78)	0.122 (0.40)
32	5706954.89	2329954.24	4.81 (15.79)	4.90 (16.06)	0.082 (0.27)
33	5706956.61	2329973.58	3.38 (11.08)	3.38 (11.10)	0.006 (0.02)
34	5706961.54	2329963.32	4.01 (13.14)	4.08 (13.38)	0.073 (0.24)
35	5706969.35	2329918.95	3.15 (10.34)	3.15 (10.34)	0 (0)
36	5706970.12	2329945.01	5.63 (18.46)	5.61 (18.41)	-0.015 (-0.05)
37	5706974.98	2329913.52	3.01 (9.87)	2.99 (9.80)	-0.021 (-0.07)
38	5706977.33	2329932.35	4.97 (16.29)	5.01 (16.44)	0.046 (0.15)
39	5706986.55	2329951.40	2.09 (6.86)	2.09 (6.86)	0 (0)
40	5706991.60	2329943.53	2.46 (8.06)	2.48 (8.14)	0.024 (0.08)
41	5706992.64	2329934.18	4.06 (13.31)	4.07 (13.34)	0.009 (0.03)
42	5706994.18	2329920.52	4.06 (13.33)	4.05 (13.29)	-0.012 (-0.04)
43	5706995.76	2329900.40	2.59 (8.49)	2.56 (8.40)	-0.027 (-0.09)

(Sheet 1 of 3)

Table 10 (Continued)					
High Point	Easting	Northing	1998 Survey El, m (ft)	2000 Survey El, m (ft)	Difference in El, m (ft)
44	5706998.01	2329908.18	3.21 (10.54)	3.19 (10.47)	-0.02 (-0.07)
45	5707003.39	2329920.05	4.35 (14.27)	3.82 (12.54)	-0.527 (-1.73)
46	5707004.81	2329930.73	3.01 (9.88)	3.10 (10.16)	0.085 (0.28)
47	5707010.10	2329917.35	3.94 (12.94)	3.92 (12.87)	-0.021 (-0.07)
48	5707010.46	2329893.18	2.71 (8.88)	2.70 (8.87)	-0.003 (-0.01)
49	5707019.69	2329884.28	3.02 (9.90)	2.96 (9.72)	-0.058 (-0.18)
50	5707023.07	2329897.66	4.48 (14.69)	4.42 (14.49)	-0.061 (-0.20)
51	2707025.53	2329910.36	4.16 (13.64)	2.84 (9.33)	-1.314 (-4.31)
52	5707031.51	2329878.36	3.96 (12.99)	3.95 (12.95)	-0.012 (-0.04)
53	5707036.81	2329890.84	4.30 (14.12)	4.34 (14.23)	0.034 (0.11)
54	5707039.35	2329881.09	3.18 (10.44)	3.19 (10.45)	0.003 (0.01)
55	5707041.57	2329898.79	4.33 (14.21)	4.40 (14.43)	0.067 (0.22)
56	5707042.79	2329871.56	2.85 (9.34)	2.85 (9.34)	0 (0)
57	5707051.18	2329901.94	2.09 (6.87)	2.09 (6.87)	0 (0)
58	5707052.32	2329886.39	4.30 (14.11)	4.27 (14.01)	-0.030 (-0.10)
59	5707054.17	2329879.60	4.06 (13.32)	4.04 (13.26)	-0.018 (-0.06)
60	5707058.36	2329867.73	4.33 (14.21)	4.30 (14.12)	-0.027 (-0.09)
61	5707064.12	2329861.39	2.97 (9.73)	3.03 (9.95)	0.067 (0.22)
62	5707065.94	2329881.23	4.40 (14.42)	4.37 (14.34)	-0.024 (-0.08)
63	5707070.51	2329886.53	3.43 (11.26)	3.42 (11.23)	-0.009 (-0.03)
64	5707072.06	2329857.45	3.79 (12.42)	3.76 (12.35)	-0.021 (-0.07)
65	5707075.14	2329866.39	4.41 (14.46)	4.40 (14.45)	-0.003 (-0.01)
66	5707078.55	2329841.29	2.42 (7.93)	2.36 (7.75)	-0.055 (-0.18)
67	5707081.24	2329875.76	4.11 (13.49)	4.10 (13.46)	-0.009 (-0.03)
68	5707091.95	2329834.79	2.43 (7.98)	2.38 (7.80)	-0.055 (-0.18)
69	5707093.64	2329845.56	4.85 (15.92)	4.76 (15.62)	-0.091 (-0.30)
70	5707098.82	2329876.11	1.91 (6.27)	1.77 (5.82)	-0.137 (-0.45)
71	57070100.79	2329853.31	4.93 (16.18)	4.90 (16.09)	-0.027 (-0.09)
72	5707108.72	2329864.92	3.97 (13.02)	3.96 (13.00)	-0.006 (-0.02)
73	5707110.63	2329835.38	4.43 (14.53)	4.43 (14.53)	0 (0)
74	5707111.10	2329824.76	3.06 (10.04)	3.05 (10.01)	-0.009 (-0.03)
75	5707113.07	2329850.36	4.88 (16.00)	4.88 (16.01)	0.003 (0.01)
76	5707122.77	2329814.97	2.71 (8.88)	2.58 (8.45)	-0.131 (-0.43)
77	5707123.01	2329864.19	2.28 (7.48)	2.27 (7.44)	-0.012 (-0.04)
78	5707130.75	2329803.07	2.14 (7.03)	2.05 (6.73)	-0.091 (-0.30)
79	5707131.61	2329843.32	4.80 (15.75)	4.46 (14.64)	-0.338 (-1.11)
80	5707131.81	2329814.68	4.18 (13.72)	3.43 (11.24)	-0.756 (-2.48)
81	5707132.40	2329834.88	4.60 (15.10)	4.59 (15.05)	-0.015 (-0.05)
82	5707132.44	2329823.79	4.56 (14.95)	4.55 (14.93)	-0.006 (-0.02)
83	5707135.75	2329853.51	2.67 (8.77)	2.65 (8.71)	-0.018 (-0.06)
84	5707139.75	2329826.71	4.63 (15.19)	4.65 (15.26)	0.021 (0.07)
85	5707147.68	2329798.07	3.35 (10.99)	3.32 (10.89)	-0.030 (-0.10)
86	5707147.90	2329834.23	4.88 (16.00)	4.86 (15.95)	-0.015 (-0.05)
87	5707156.17	2329844.24	2.18 (7.16)	2.12 (6.97)	-0.058 (-0.19)
88	5707163.87	2329832.65	3.63 (11.92)	3.61 (11.85)	-0.021 (-0.07)

(Sheet 2 of 3)

Table 10 (Concluded)					
High Point	Easting	Northing	1998 Survey El, m (ft)	2000 Survey El, m (ft)	Difference in El, m (ft)
89	5707166.65	2329793.60	5.17 (16.95)	5.07 (16.62)	-0.101 (-0.33)
90	5707170.38	2329818.07	4.62 (15.17)	4.62 (15.17)	0 (0)
91	5707172.52	2329803.11	4.91 (16.10)	4.92 (16.13)	0.009 (0.03)
92	5707179.66	2329790.68	4.88 (16.02)	4.82 (15.81)	-0.064 (-0.21)
93	5707183.03	2329829.10	1.96 (6.44)	2.05 (6.72)	0.085 (0.28)
94	5707195.10	2329776.67	4.82 (15.82)	4.80 (15.76)	-0.018 (-0.06)
95	5707195.42	2329828.13	1.81 (5.94)	1.81 (5.94)	0 (0)
96	5707202.80	2329797.94	5.15 (16.88)	5.07 (16.65)	-0.070 (-0.23)
97	5707217.11	2329808.24	3.22 (10.55)	3.20 (10.51)	-0.012 (-0.04)
98	5707219.42	2329793.57	5.25 (17.23)	5.08 (16.68)	-0.168 (-0.55)
99	5707229.90	2329768.01	4.75 (15.60)	4.75 (15.60)	0 (0)
100	5707236.12	2329791.59	4.27 (14.00)	4.23 (13.89)	-0.034 (-0.11)
101	5707241.33	2329801.74	3.43 (11.24)	3.43 (11.24)	0 (0)
102	5707242.56	2329779.52	5.11 (16.75)	5.10 (16.72)	-0.009 (-0.03)
103	5707257.78	2329762.78	5.19 (17.02)	5.18 (16.98)	-0.012 (-0.04)
104	5707259.83	2329792.94	3.78 (12.39)	3.74 (12.27)	-0.037 (-0.12)
105	5707263.17	2329803.69	2.48 (8.13)	2.46 (8.08)	-0.015 (-0.05)
106	5707265.48	2329781.37	5.23 (17.17)	5.17 (16.95)	-0.067 (-0.22)
107	5707273.30	2329774.88	5.53 (18.15)	5.36 (17.58)	-0.174 (-0.57)
108	5707277.27	2329805.70	1.72 (5.63)	1.69 (5.56)	-0.021 (-0.07)
109	5707280.77	2329754.23	3.74 (12.27)	3.74 (12.27)	0 (0)
110	5707281.21	2329789.89	4.80 (15.75)	4.82 (15.83)	0.024 (0.08)
111	5707282.52	2329798.78	2.05 (6.74)	2.06 (6.77)	0.009 (0.03)
112	5707285.96	2329765.73	5.19 (17.03)	5.12 (16.80)	-0.070 (-0.23)
113	5707292.15	2329776.62	5.09 (16.70)	5.09 (16.70)	0 (0)
114	5707300.17	2329760.23	5.25 (17.21)	5.22 (17.12)	-0.027 (-0.09)
115	5707302.52	2329800.20	2.25 (7.37)	2.23 (7.33)	-0.012 (-0.04)
116	5707304.85	2329783.57	5.02 (16.47)	4.99 (16.36)	-0.034 (-0.11)
117	5707311.45	2329744.39	2.62 (8.61)	2.60 (8.54)	-0.021 (-0.07)
118	5707313.49	2329770.79	5.13 (16.84)	5.09 (16.70)	-0.043 (-0.14)
119	5707314.71	2329753.73	4.26 (13.99)	4.26 (13.99)	0 (0)
120	5707323.43	2329791.95	3.60 (11.81)	3.52 (11.56)	-0.076 (-0.25)
121	5707333.04	2329768.72	5.30 (17.39)	5.29 (17.37)	-0.006 (-0.02)
122	5707334.69	2329782.03	4.82 (15.80)	4.80 (15.76)	-0.012 (-0.04)

(Sheet 3 of 3)

3 Conclusions and Recommendations

Conclusions

The monitoring plan for Morro Bay Harbor entrance channel was formulated to test five specific hypotheses about the redesigned Federal project, as discussed in Chapter 2. The monitoring objective was to determine if nonstructural modifications at the harbor entrance are performing as predicted. Conclusions relative to each hypothesis are presented in this section.

Hypothesis a: Improvements constructed at Morro Bay entrance in December 1995 will result in significantly improved navigation conditions in harbor entrance

The monitoring study did not include quantitative data collection relative to this hypothesis. It was impractical to place a gauge in the deepened entrance, since it would be a hazard to navigation and/or would be buried by shoaling. However, the Morro Bay harbor master's office reports that hazardous breaking wave conditions in the deepened entrance occur significantly less often than in the preproject condition. This information, coupled with survey data showing that a deepened entrance was maintained during the monitoring period, leads to the conclusion that hypothesis a has proven to be true.

Hypothesis b: Improvements will have no negative impact on existing structures

Existing structures in the vicinity of the modified entrance include the south and north breakwaters. Physical model tests showed that the south breakwater head could experience a more severe wave climate with the modified entrance in place, though the increase in severity was small. Based on the design conditions, the south breakwater was predicted to be unaffected by the increase in wave heights. Photogrammetric surveys of the above-water portion of the south breakwater show no significant changes over a 2-year monitoring period (1998-2000). The north breakwater was predicted to be unaffected by the project, so it was not subjected to detailed monitoring. No significant changes to the north breakwater were reported or observed during the monitoring period.

This hypothesis is concerned with structural design based on return periods of up to 50 years. Thus, a short-term monitoring study cannot be expected to be conclusive. No episodic storms occurred during monitoring and no significant structural impact was observed. Therefore, this hypothesis was supported, but not conclusively proven, by the monitoring study.

Hypothesis c: Improvements can be effectively maintained with a 3-year dredging interval in entrance

Dredging intervals during the monitoring study were considerably shorter than the 3-year design interval. The longest interval was 15 months and all others were less than 10 months. Sediment volume stored in the entrance and channel was computed for each survey, as was shown in Figure 25. The volume predicted by USAED, Los Angeles (1994) for removal during initial dredging was 684,300 cu m (895,000 cu yd), and 752,980 cu m (984,800 cu yd) were actually dredged. None of the subsequent dredging episodes re-established design advance depth in the entrance, channel, and sediment trap, which would have shown as a zero volume in the figure. The stored volume generally increased between successive surveys except when dredging occurred.

The rate of volume increase varied during the nearly 6-year time period since the project was initiated, but a fairly consistent trend was evident. The annual sedimentation rate predicted in the feasibility report was 183,500 cu m/year (240,000 cu yd/year), corresponding to a monthly rate of 15,300 cu m/month (20,000 cu yd/month) (USAED, Los Angeles, 1991). This predicted rate is bracketed by measured rates for the longer intervals. The advance maintenance and sand trap sediment storage capacity of the final project, designed for a 3-year maintenance cycle, was predicted by USAED, Los Angeles (1994) as 450,000 cu m (590,000 cu yd), corresponding to a monthly rate of 12,500 cu m/month (16,400 cu yd/month). This predicted rate is lower than rates computed for most survey intervals during monitoring, but higher than the computed rate for the longest survey interval. Even the longest survey/dredging interval was considerably shorter than the 3-year cycle. Overall, in comparison to shoaling volumes and rates from survey data shown in Figures 25 and 26, the Los Angeles District-predicted shoaling over a 3-year time period appears consistent with project experience.

The survey data indicate that infilling could be more rapid during unusually stormy winters, as was shown for the winter of 1997-98. To consistently preserve benefits of the deepened entrance, the monitoring study indicates that the 3-year dredging interval is a maximum.

Hypothesis d: Model investigations accurately quantified wave conditions in entrance and correctly defined sediment patterns and deposition in qualitative sense

Numerical and physical modeling studies quantified transformation of various incident wave conditions in the preproject and with project harbor entrances. Wave gauges operated in the monitoring study provide data on incident waves (Gauge CA002) and transformed waves at two locations within the entrance. These data were used to evaluate the accuracy of model studies. For practical reasons, gauges could not be placed in the immediate vicinity of the breakwater gap, where wave height decreases sharply with distance inside the gap. However, gauge data collected outside the entrance gap and in the main channel were sufficient to show that physical model wave data through the entrance are accurate representations of the prototype. The field data also reveal that the original numerical model investigation significantly overestimated wave heights between the breakwater gap and groin. Present numerical model technology, which is now used in place of the technology applied at Morro Bay Harbor, is demonstrated to be comparable to physical model and field data and much improved over the original numerical model study.

Physical model studies also predicted wave-driven sediment transport paths and deposition patterns. Incident wave directions of 250-deg and 300-deg azimuth were tested to examine northward and southward transport patterns. Prototype shoaling patterns are typically qualitatively similar to those observed in the physical model for 250-deg wave directions. Sediment moves into the modified entrance from the south and a transport path continues around the south breakwater head into the sand trap area. Sediment infilling from the north side of the modified entrance was predicted by physical modeling of the 300-deg wave direction, but not clearly observed in the prototype data.

Prototype incident wave data from Morro Bay Gauge CA002 show that 250 deg is a fairly typical direction. Mean wave direction is 265 deg. Twenty-five percent of the waves recorded came from directions between 208 deg and 258 deg and 25 percent came from directions between 258-268 deg. Thus, physical model tests with 250-deg wave direction are quite representative of prototype waves from the south. For the more northerly half of prototype observations, 25 percent came from 268-275 deg and 25 percent from 275-296 deg. Thus, prototype wave data reveal for the time period monitored that the 300-deg direction tested in the physical model may have been quite extreme.

Prototype sediment deposition patterns, as represented in survey data, result from many different wave conditions occurring over the survey interval. They are also modified by tidal currents and wind effects, neither of which were included in physical modeling of sediment transport and deposition patterns. However, the qualitative deposition patterns predicted in physical modeling of 250-deg wave directions should be appropriate for prototype conditions and they are well supported by prototype data.

Hypothesis e: Methodology used in determining sedimentation rates in harbor entrance was valid based on field data, model predictions, and sound engineering judgment

Prediction of sedimentation rates in the harbor entrance was a difficult, but crucial, element of project design. Based on incident wave information available for design, the net potential longshore transport was strongly southward. Prototype incident wave data yield the same conclusion. However, northward transport was recognized in design as dominant in the process of harbor entrance shoaling. Prototype data also support this conclusion. Sedimentation rates for planning and design were estimated using historic dredging data, model predictions about the impact of the widened, deepened dredged area on shoaling rate, and engineering judgment based on experience at this and other coastal sites.

The prototype incident directional wave gauge deployed in the monitoring study operated over a relatively short time period due to equipment problems. Directional waves incident to Morro Bay Harbor were reconstructed with reasonable confidence using deepwater gauges up- and downcoast from Morro Bay. The data consistently indicate that southward potential longshore transport is around an order of magnitude greater than northward transport. Wave-driven northward potential longshore transport is considerably less than observed shoaling rates. Over the longer survey intervals, gauge data yield annual potential longshore transport rates of around 600,000 cu m/year (800,000 cu yd/year) southward and 48,000 cu m/year (63,000 cu yd/year) northward. For design, USAED, Los Angeles District (1994) reduced the computed southward potential longshore transport to account for blocking by Morro Rock, giving an effective southward transport of 54,300 cu m/year (71,000 cu yd/year). Northward transport was determined as 305,800 cu m/year (400,000 cu yd/year). Recognizing that calculations involve assumptions and approximations, monitoring study data do not appear to support previous calculations of potential longshore sediment transport.

Although wave-driven longshore transport is certainly a key source of energy for sediment transport in Morro Bay Harbor entrance, this forcing is not so directly related to transport quantities and directions as along a straight coastline. The critical design result was overall sedimentation rate in the dredged project area, since that determines maintenance requirements and costs. As discussed under hypothesis c, the design methodology resulted in a predicted sedimentation rate which is very reasonable in comparison to shoaling rates observed during monitoring. Thus, it can be concluded that, overall, hypothesis e is supported by monitoring results.

Recommendations

The Morro Bay Harbor project shoaling volumes and rates should continue to be monitored by the Los Angeles District. Results from the present study give a baseline for interpreting future shoaling data, but data on project behavior over a longer time period would be valuable. Such data would be especially important if the project is operated to achieve the fully-dredged design depths with a 3-year target interval between dredging.

The relationship between incident waves, currents, and other possible natural forces on sedimentation processes at Morro Bay Harbor entrance is still not clearly understood. Further measurement and model studies are needed for a more complete understanding of this complex coastal area. An intensive field study over a short time period (such as one month) with some storm activity and corresponding modeling could be especially helpful.

Even though no significant changes occurred for above-water armor stone on the south breakwater during the relatively short monitoring period, the structure should still continue to be monitored on a periodic basis and inspected after major storm events.

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Appendix A
Bathymetric Survey Contour Plots

This appendix presents contour plots from all bathymetric surveys of Morro Bay entrance taken between September 1995 and August 2000. Depths are in meters referenced to mean lower low water (mllw).

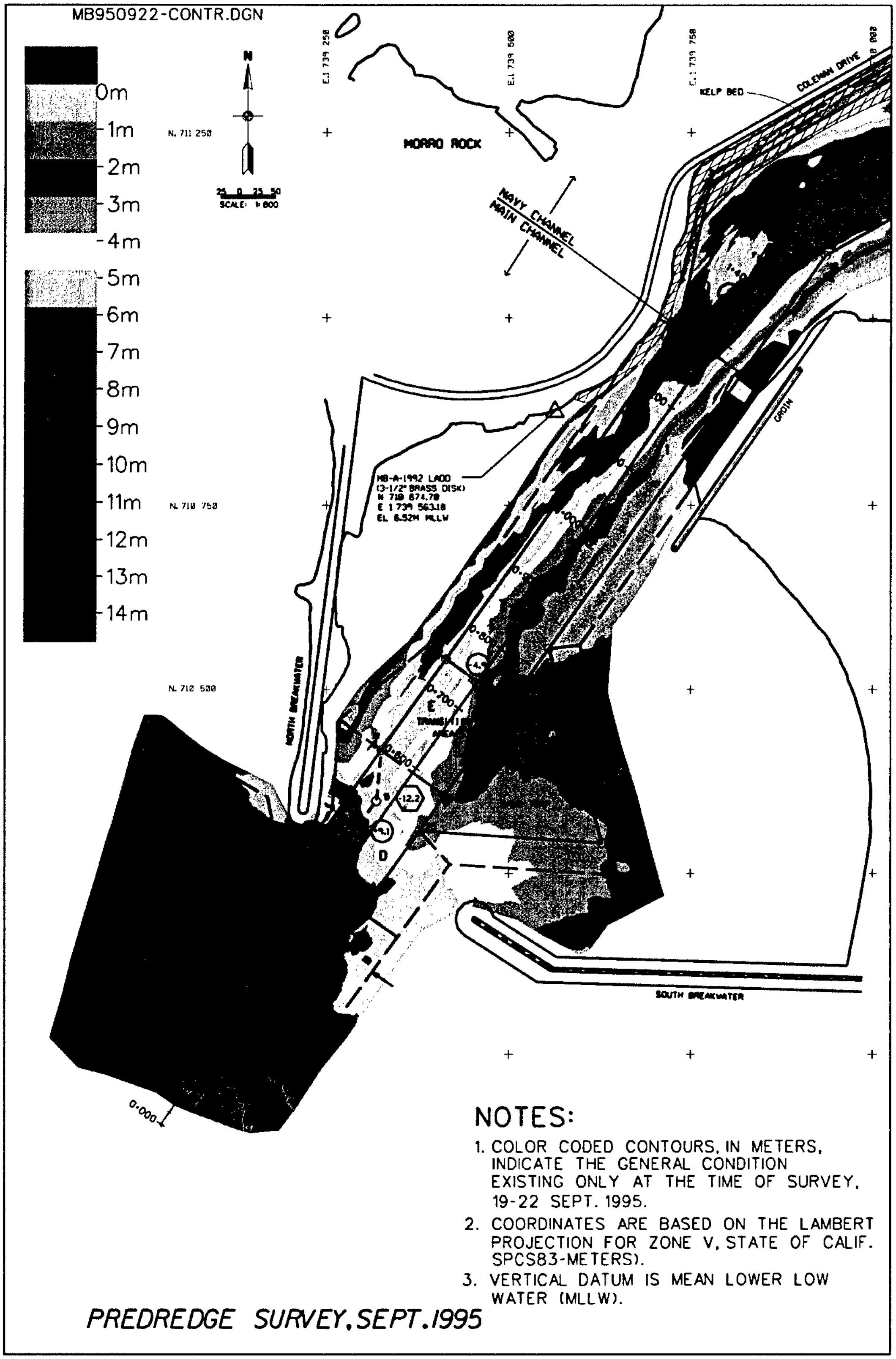


Figure A1. Bathymetry at Morro Bay entrance, 19-22 September 1995

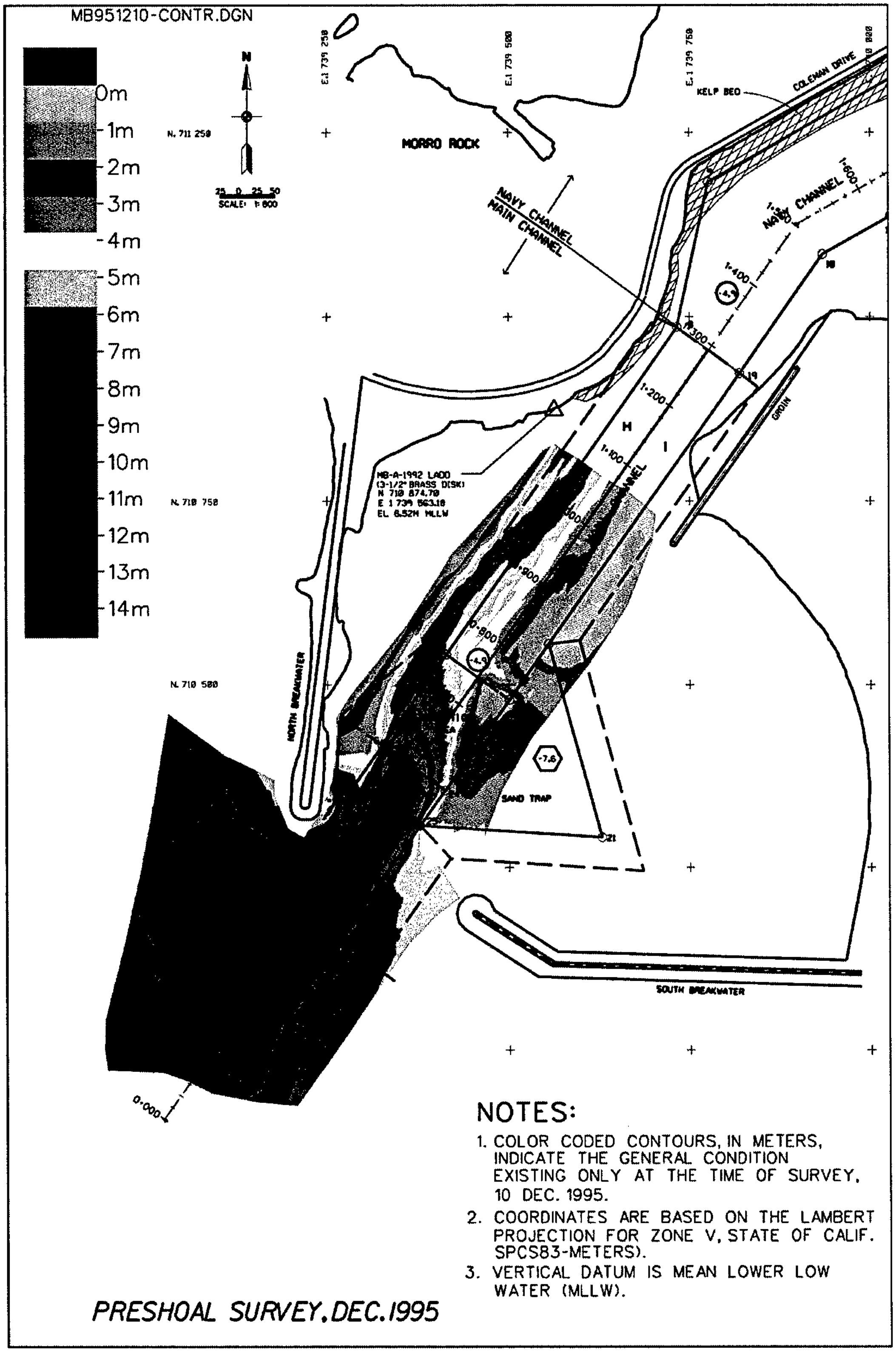


Figure A2. Bathymetry at Morro Bay entrance, 10 December 1995

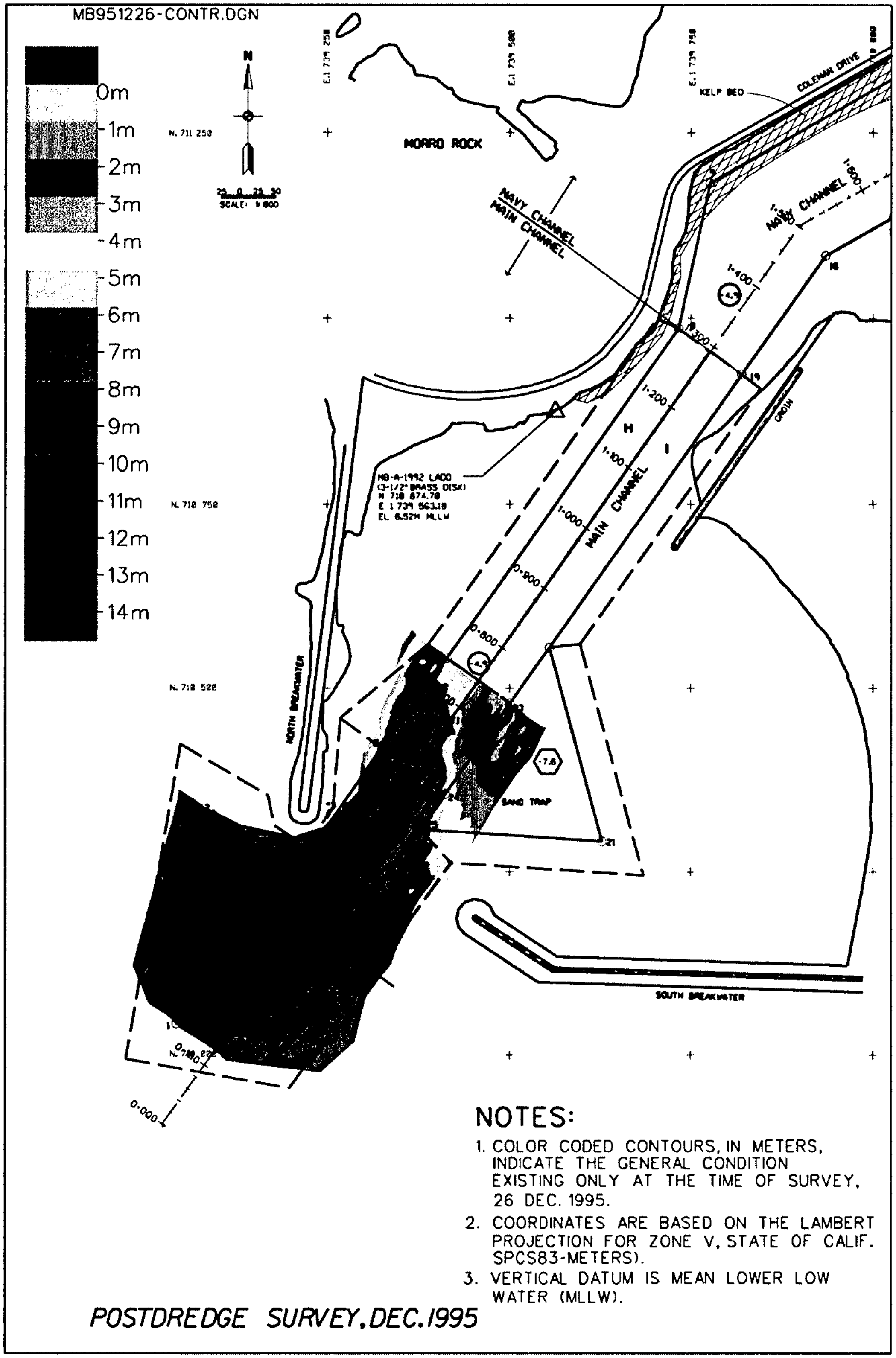


Figure A3. Bathymetry at Morro Bay entrance, 26 December 1995

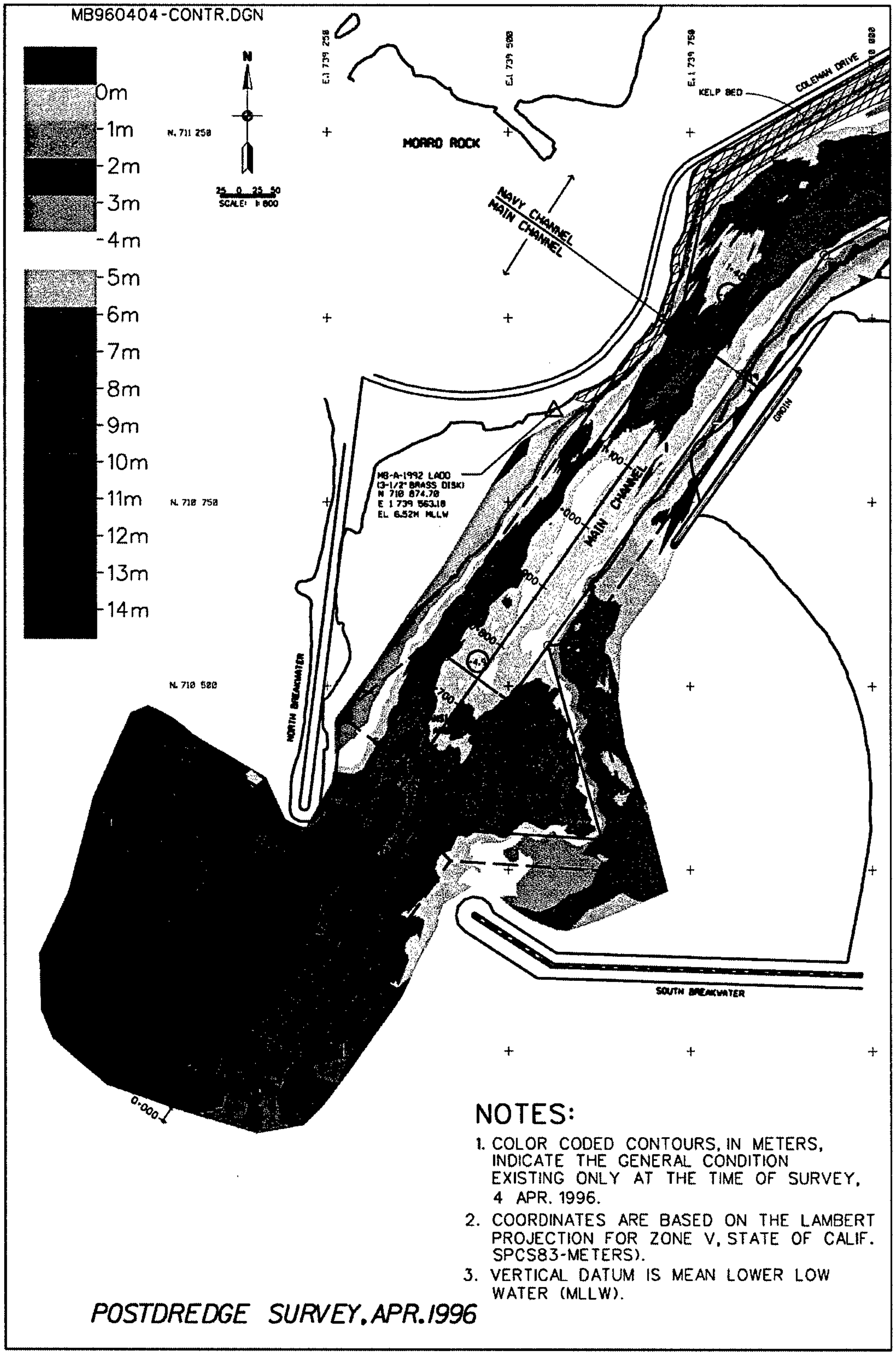


Figure A4. Bathymetry at Morro Bay entrance, 4 April 1996

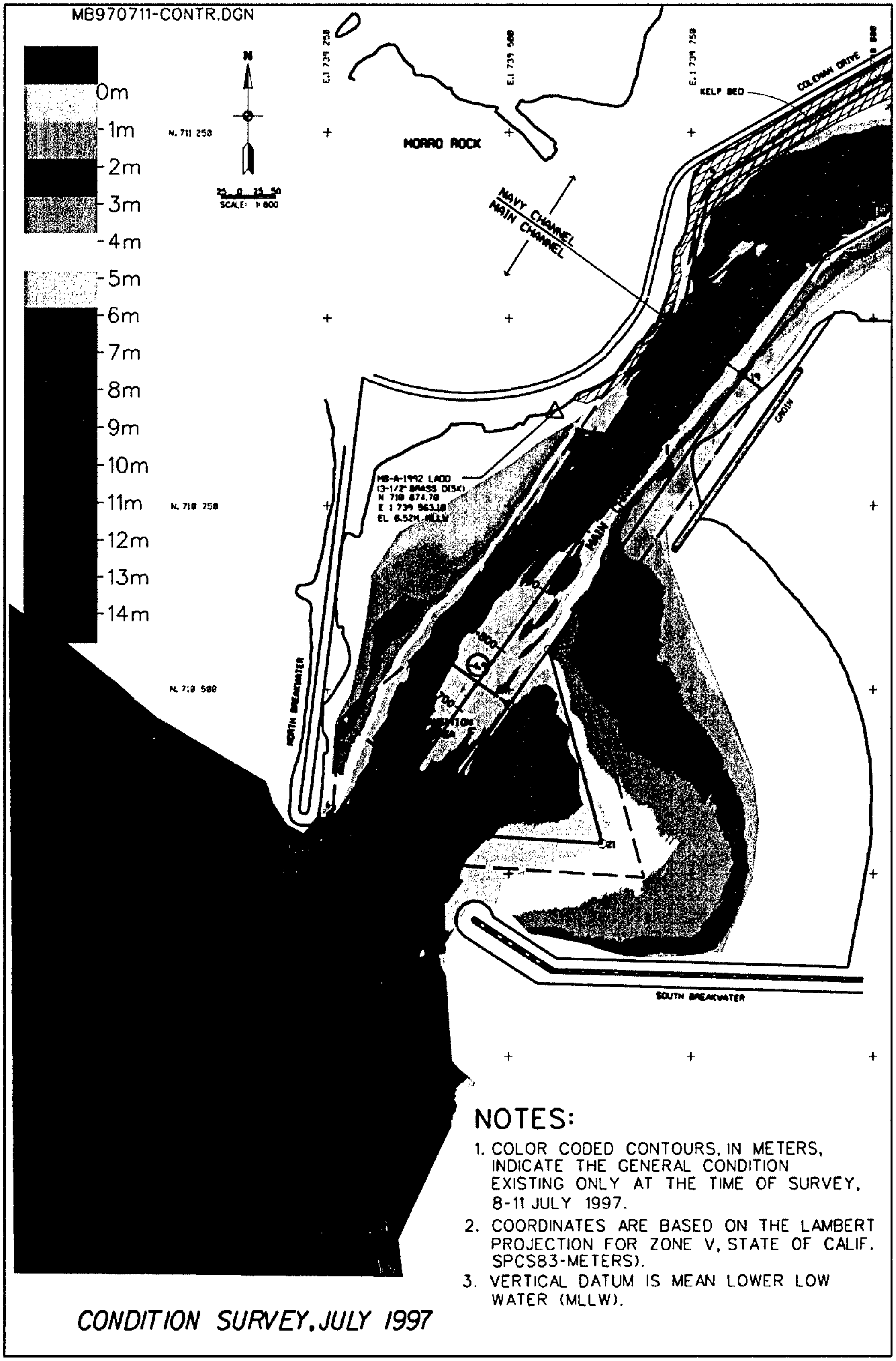


Figure A5. Bathymetry at Morro Bay entrance, 8-11 July 1997

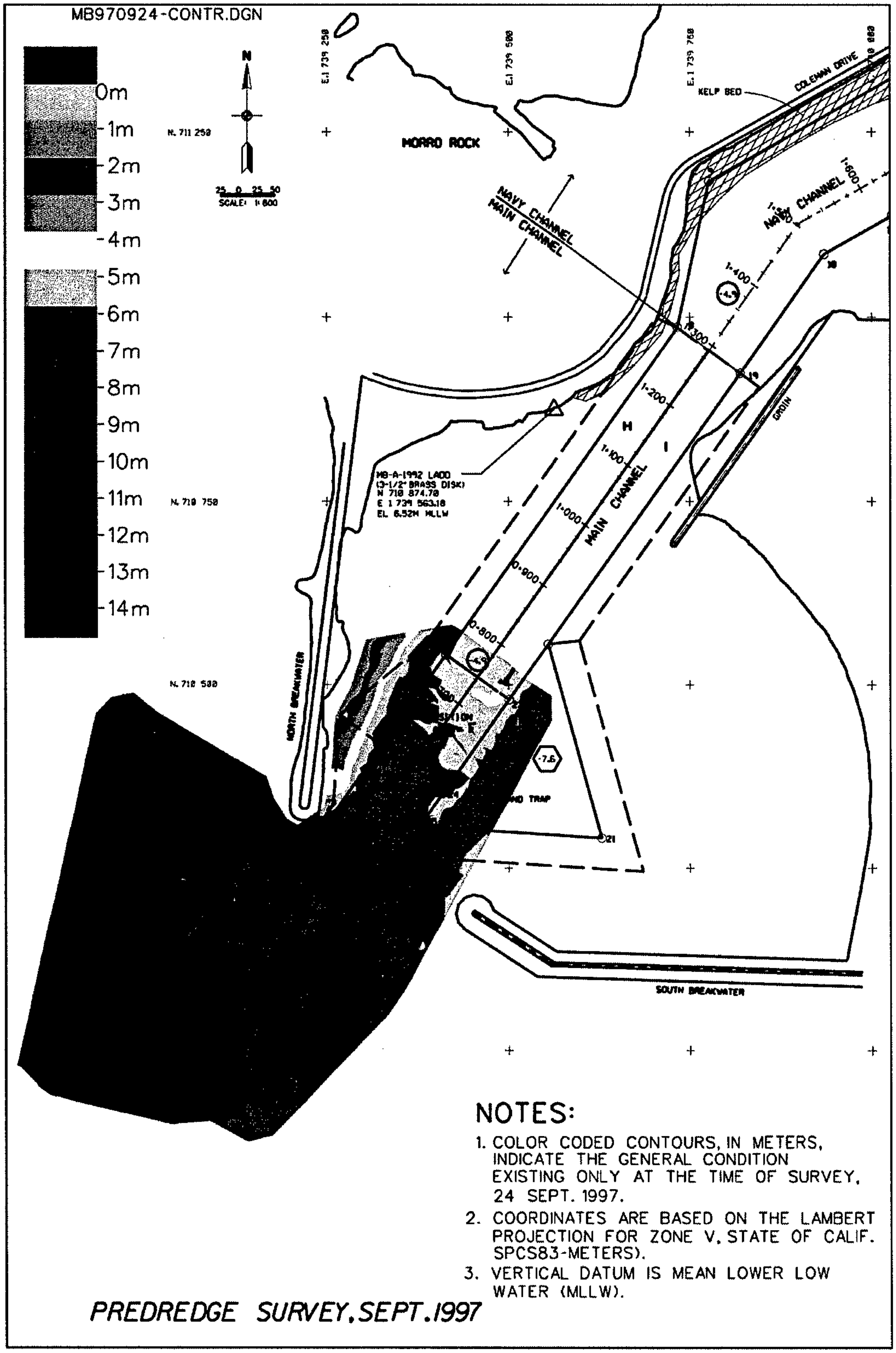


Figure A6. Bathymetry at Morro Bay entrance, 24 September 1997

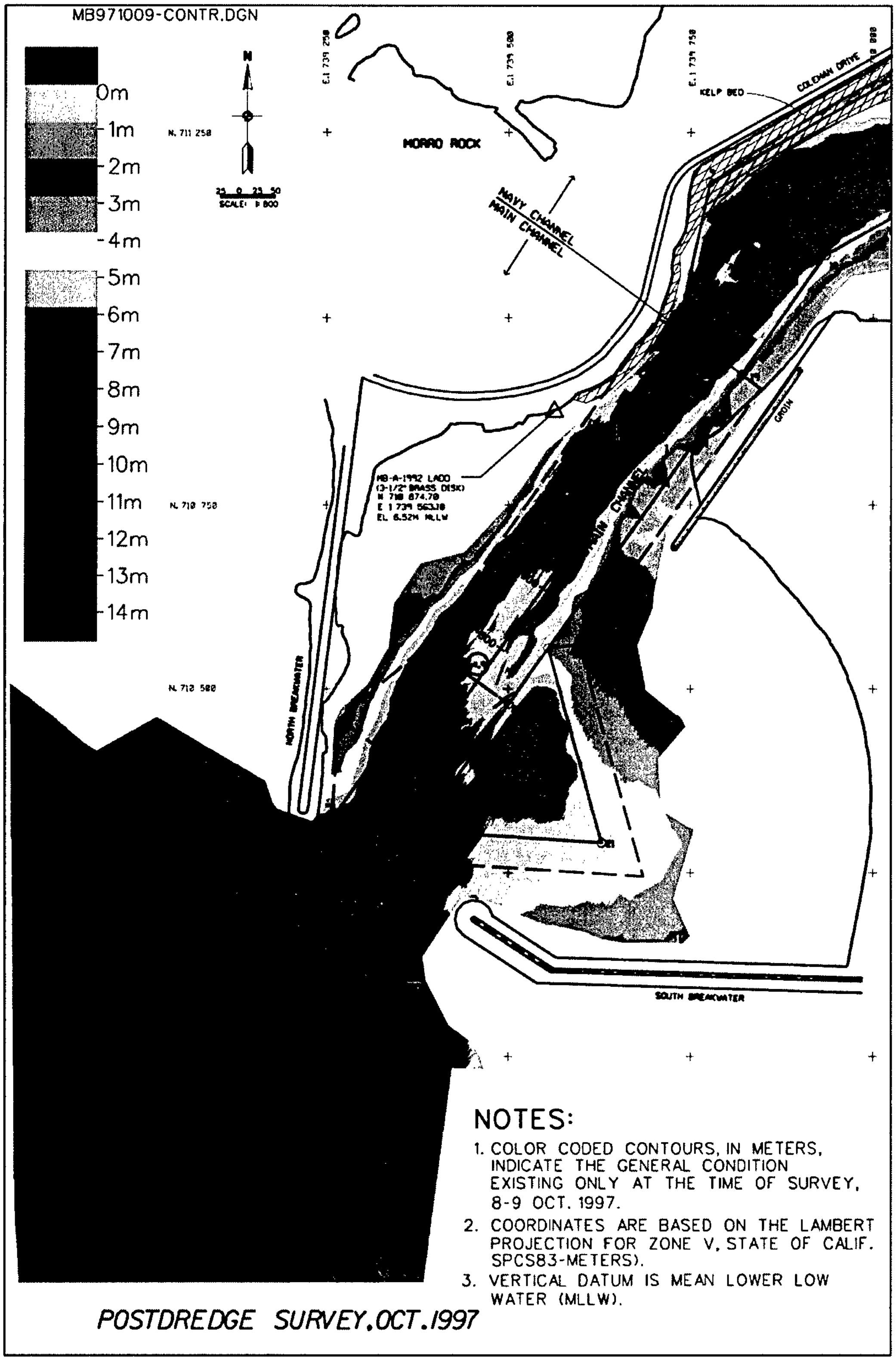


Figure A7. Bathymetry at Morro Bay entrance, 8-9 October 1997

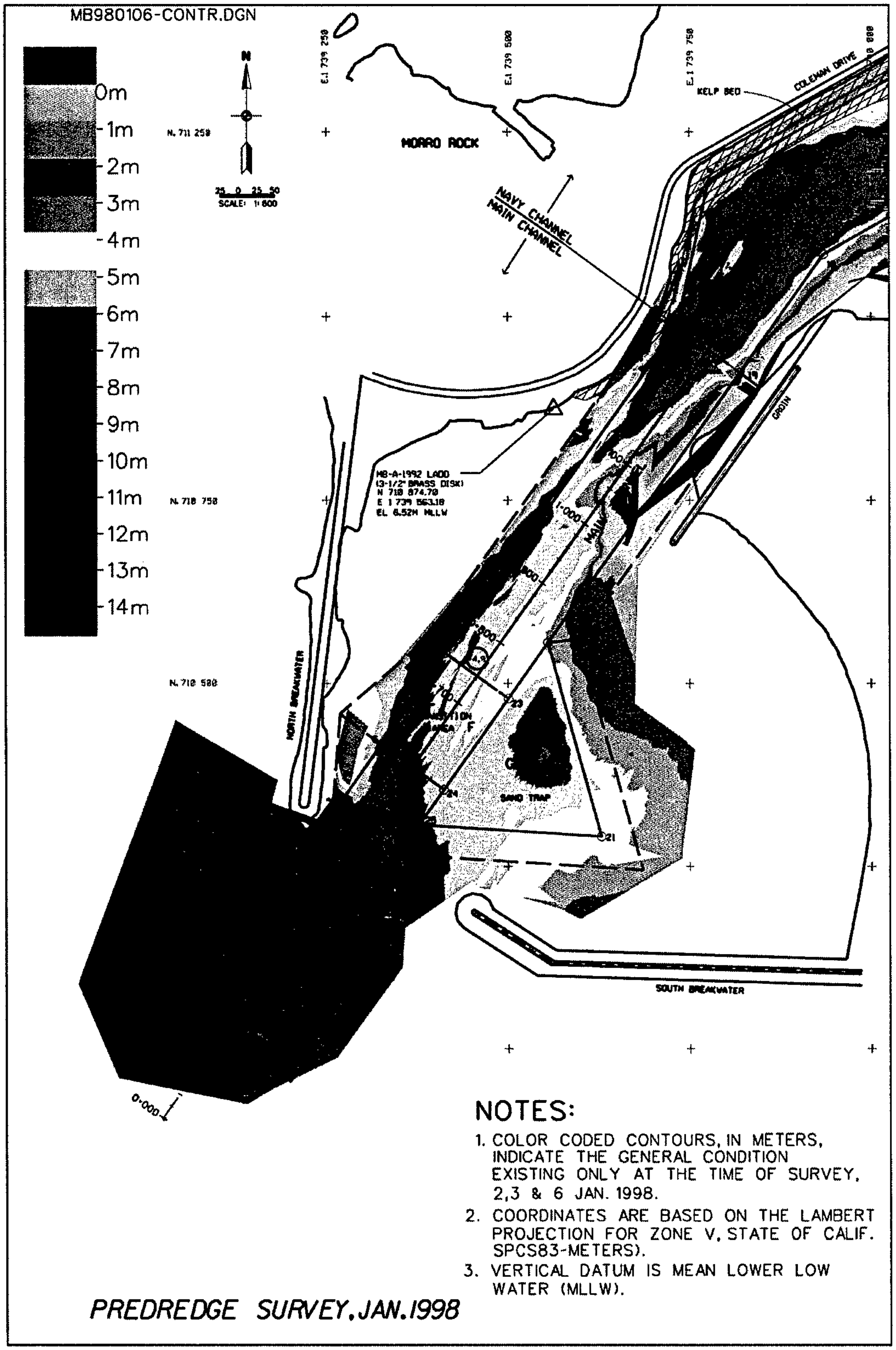


Figure A8. Bathymetry at Morro Bay entrance, 2-6 January 1998

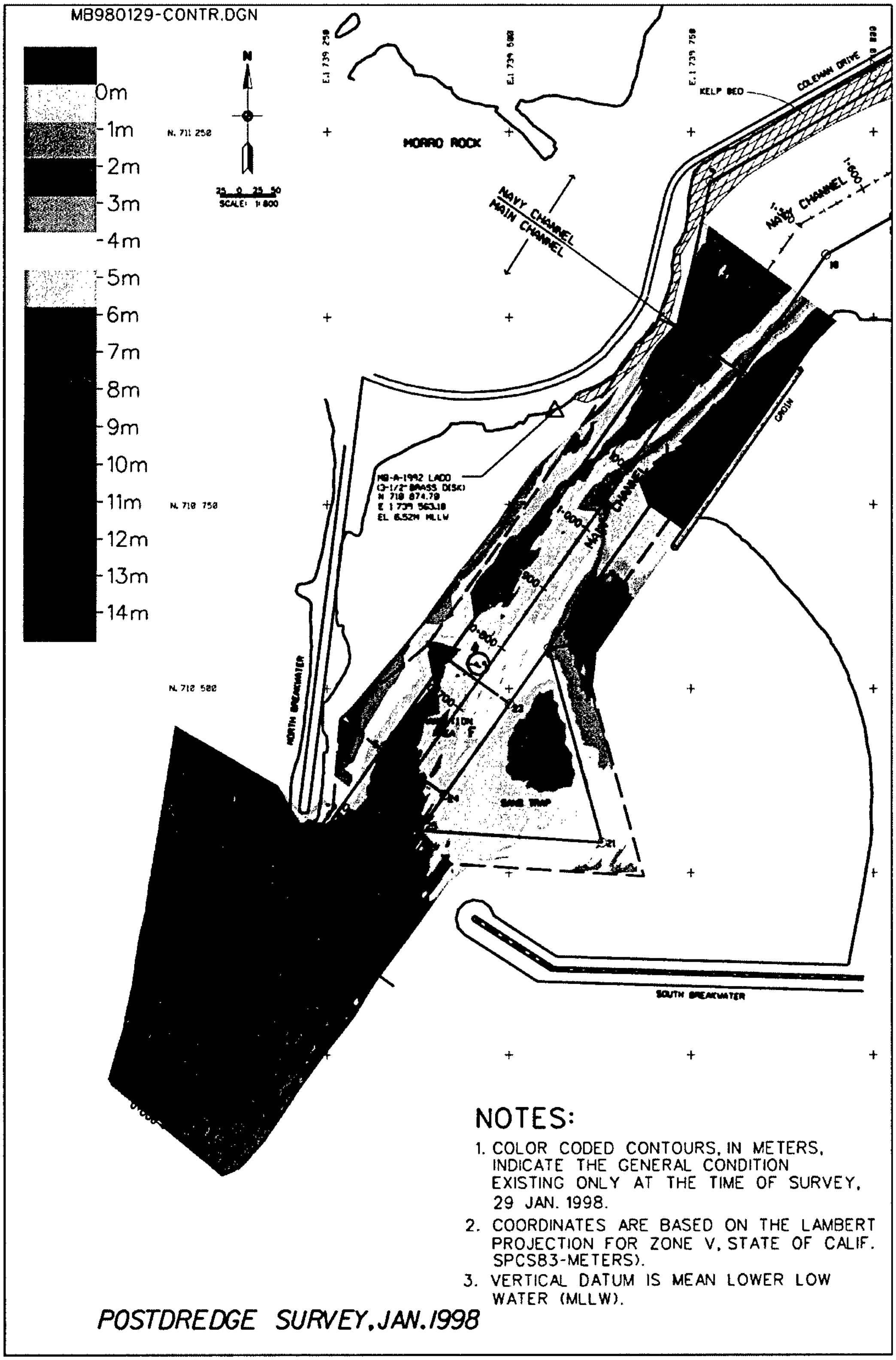


Figure A9. Bathymetry at Morro Bay entrance, 29 January 1998

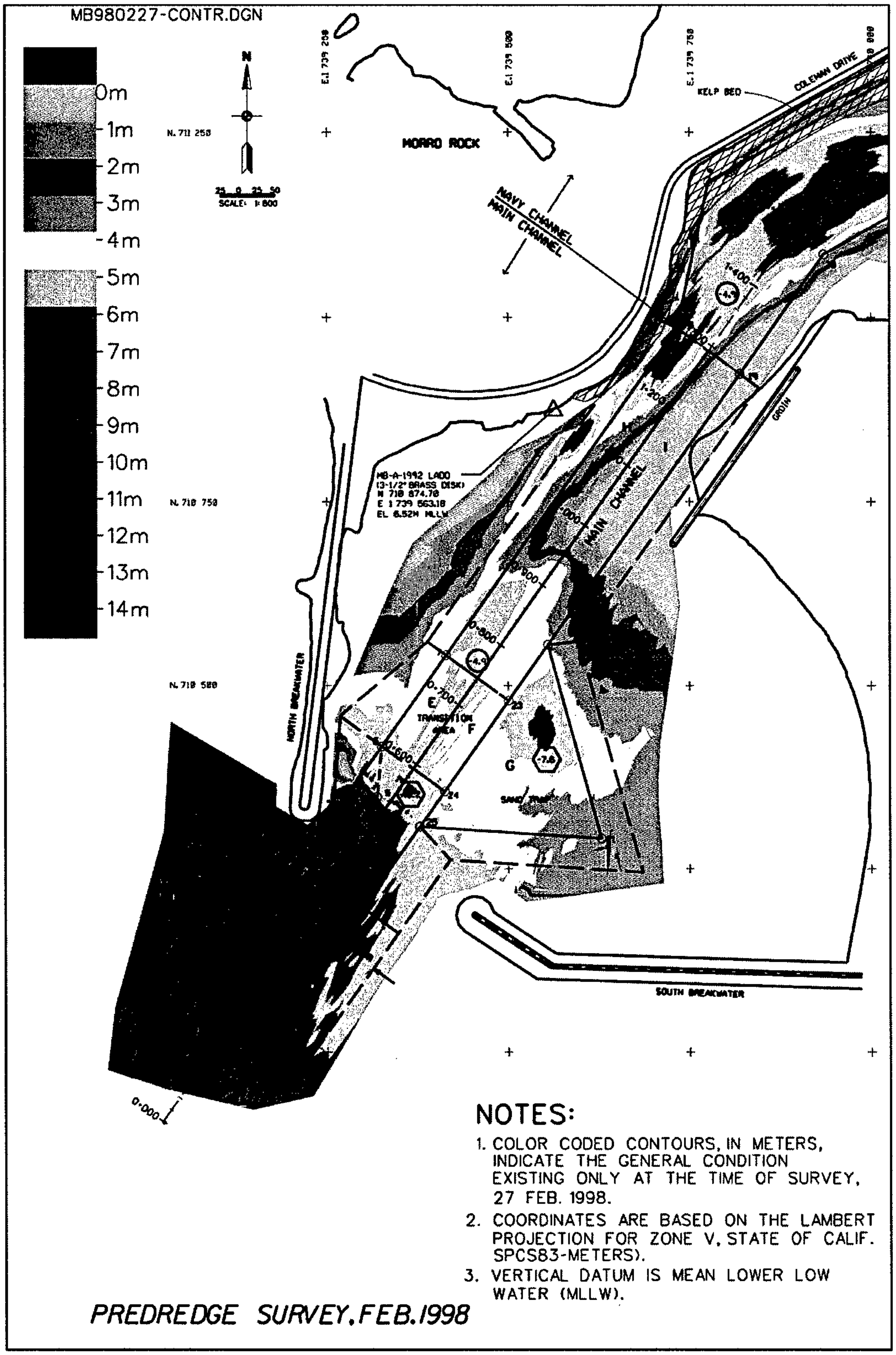


Figure A10. Bathymetry at Morro Bay entrance, 27 February 1998

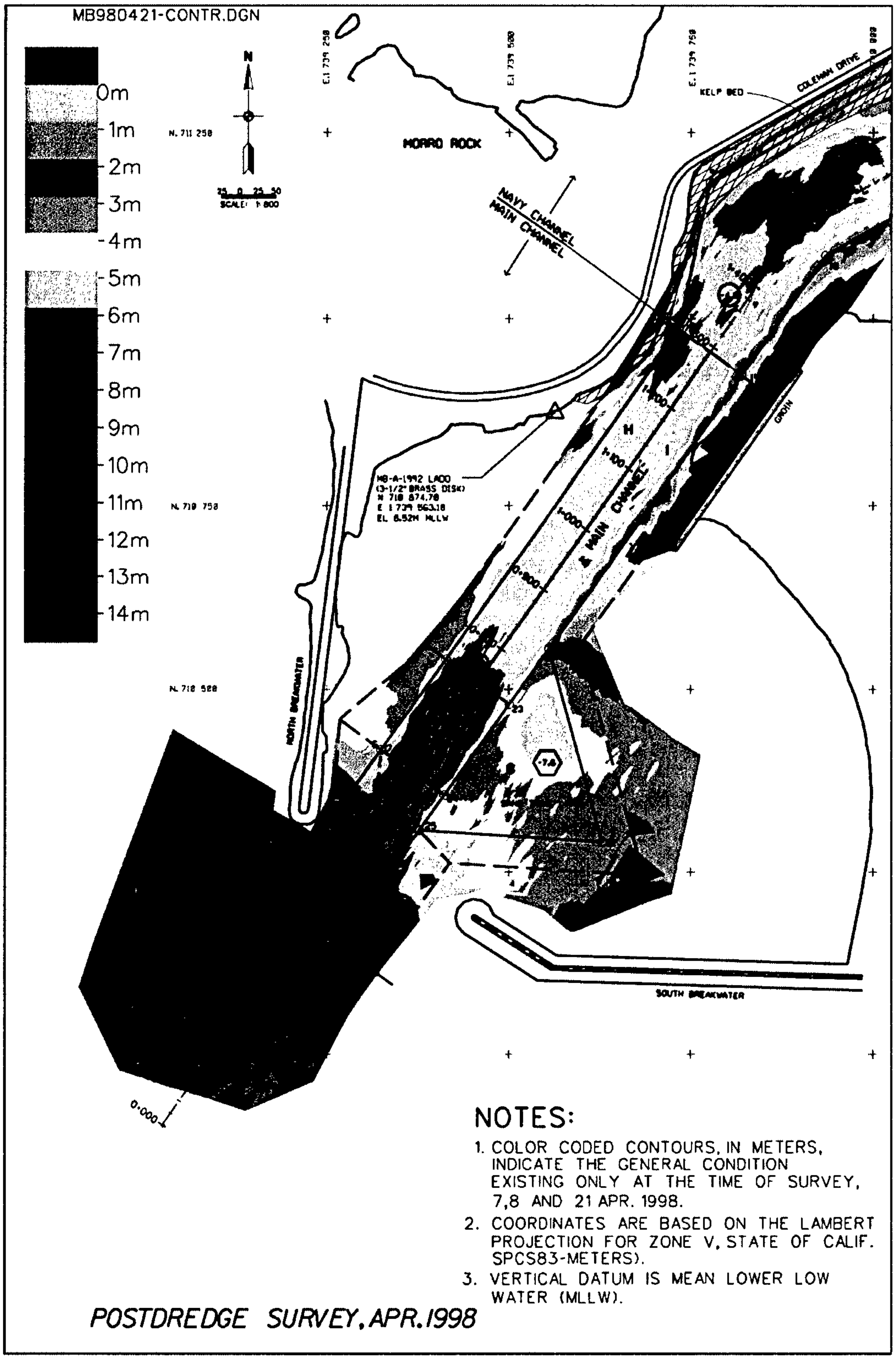


Figure A11. Bathymetry at Morro Bay entrance, 7-8 and 21 April 1998

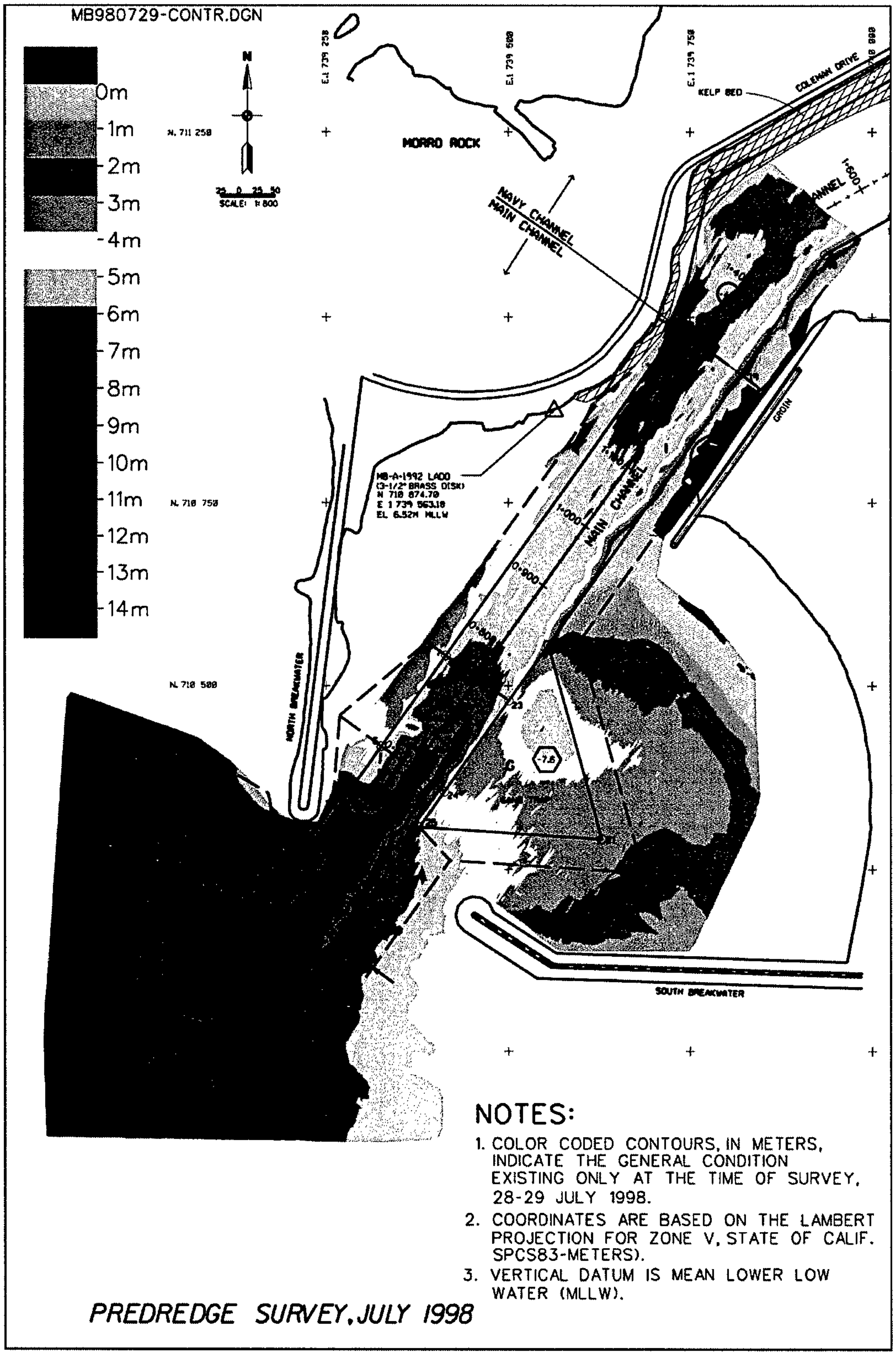


Figure A12. Bathymetry at Morro Bay entrance, 28-29 July 1998

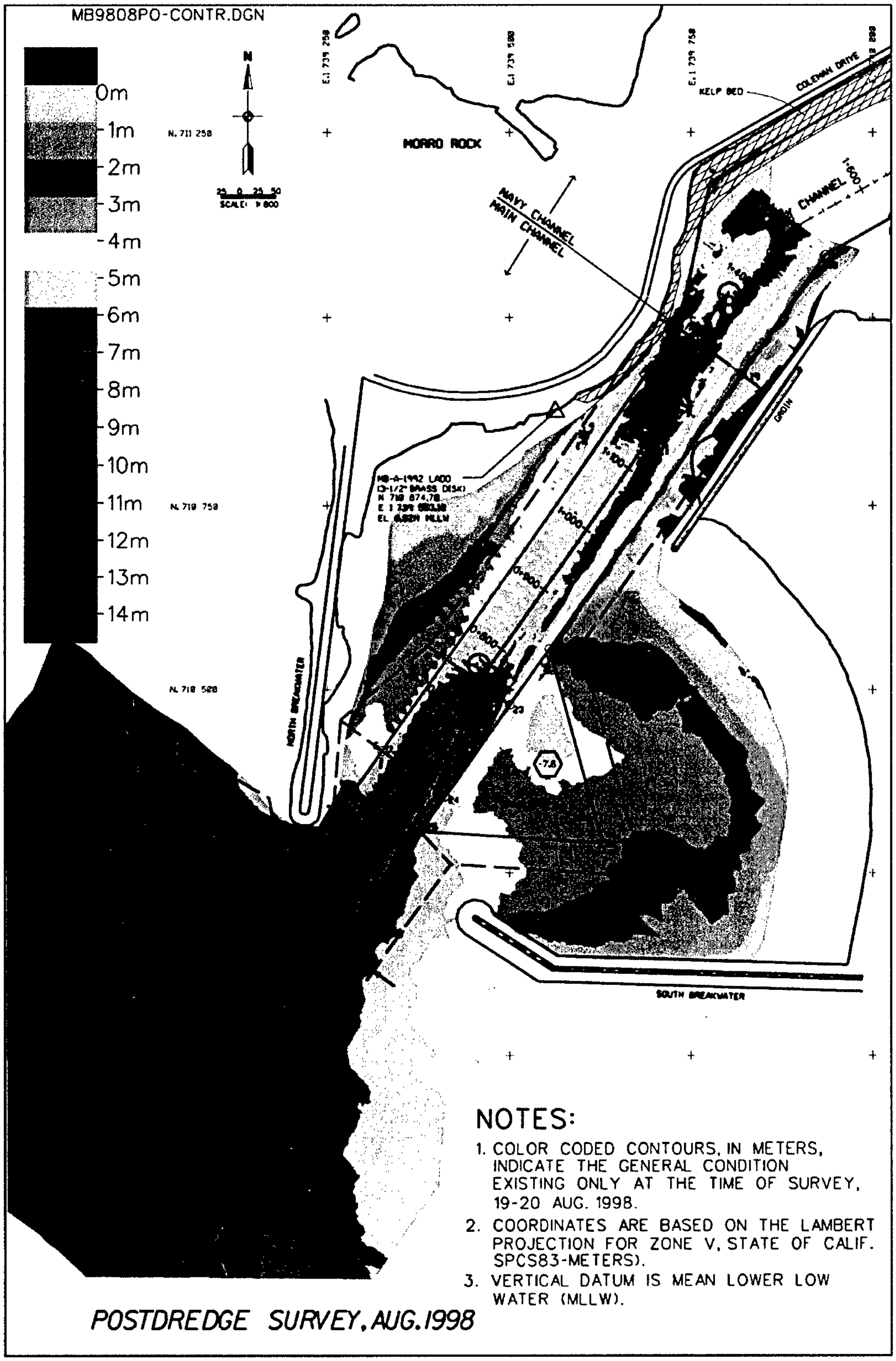


Figure A13. Bathymetry at Morro Bay entrance, 19-20 August 1998

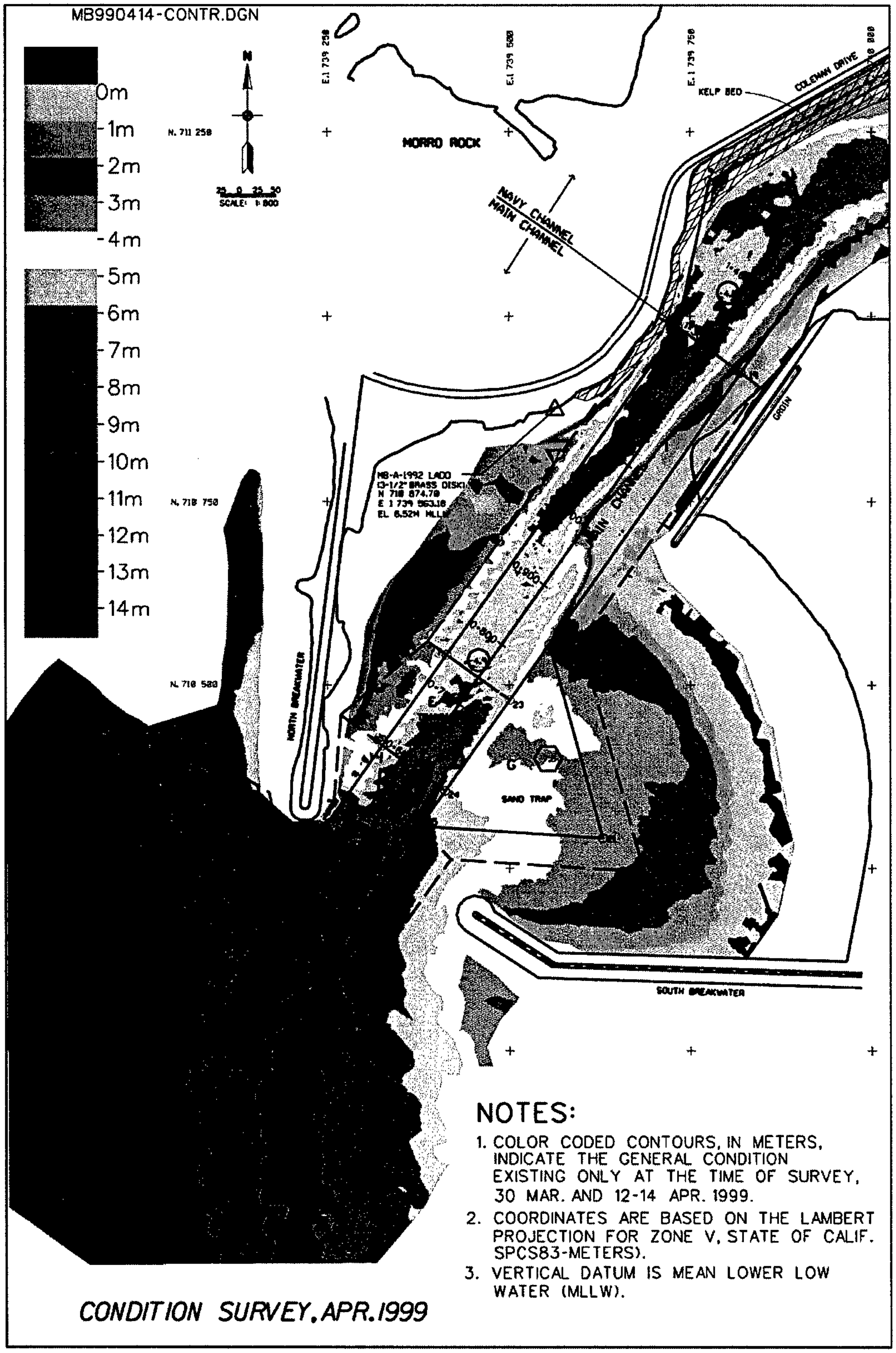


Figure A14. Bathymetry at Morro Bay entrance, 30 Mar and 12-14 April 1999

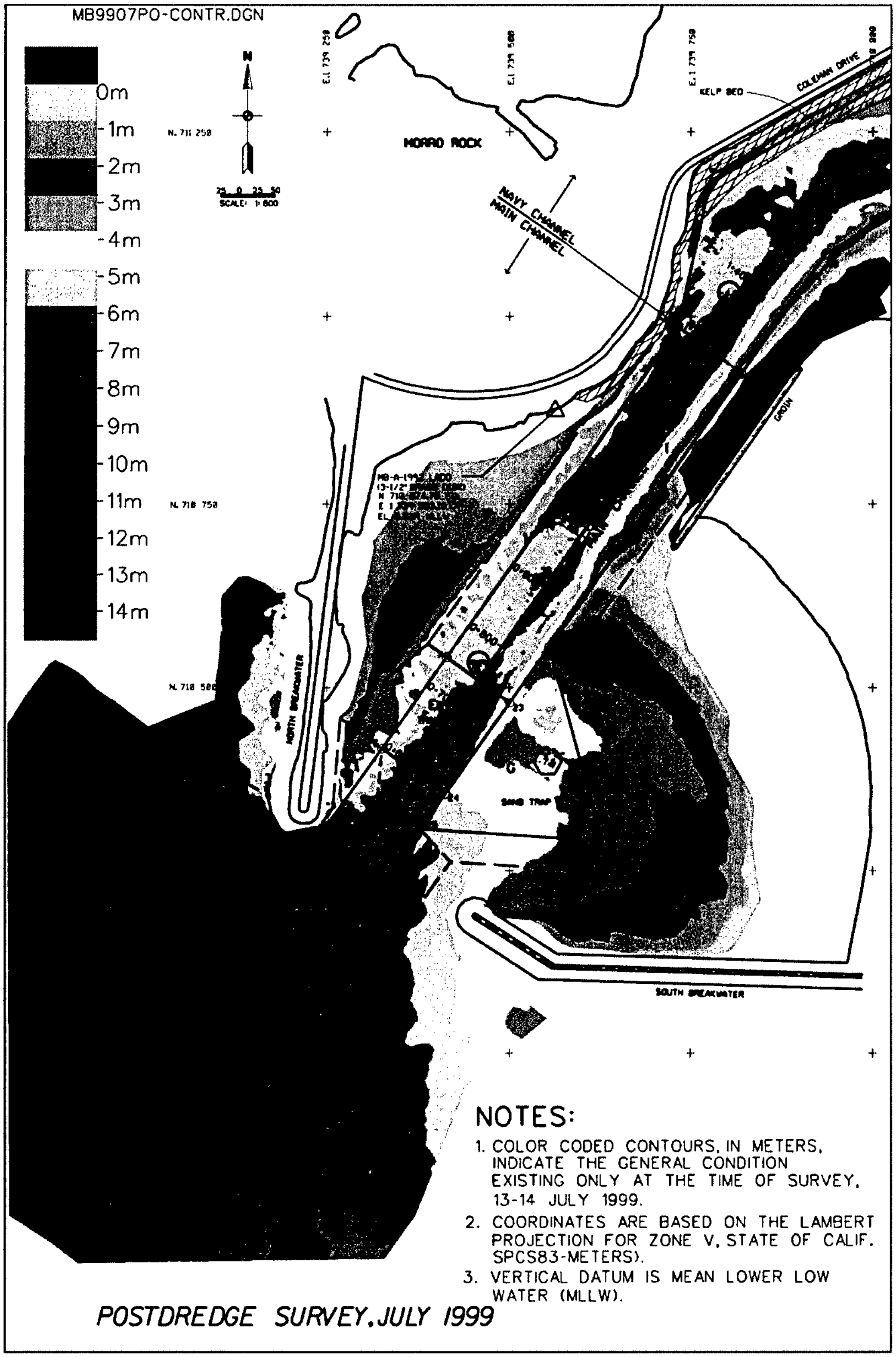


Figure A15. Bathymetry at Morro Bay entrance, 13-14 July 1999

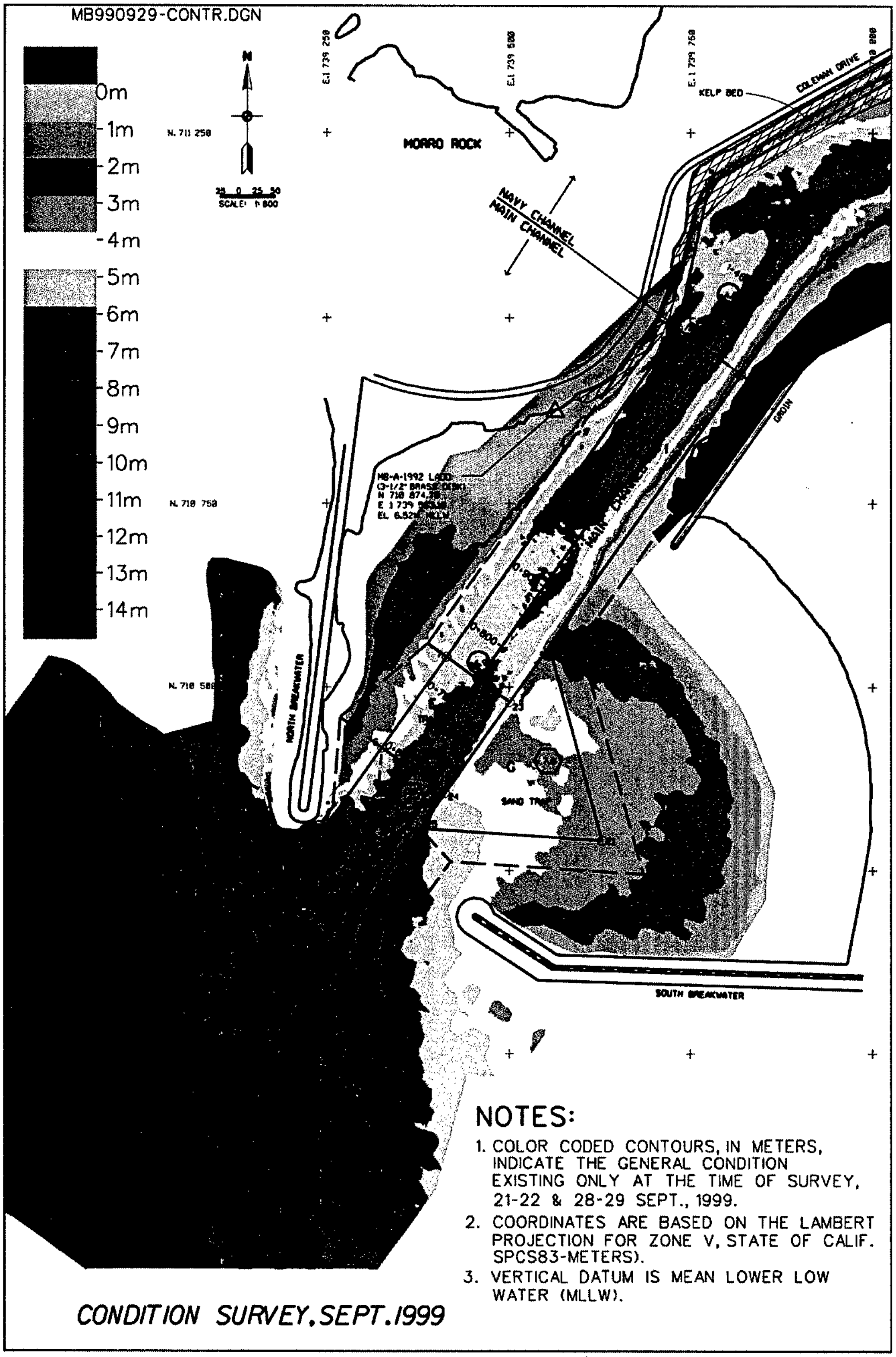


Figure A16. Bathymetry at Morro Bay entrance, 21-22 and 28-29 September 1999

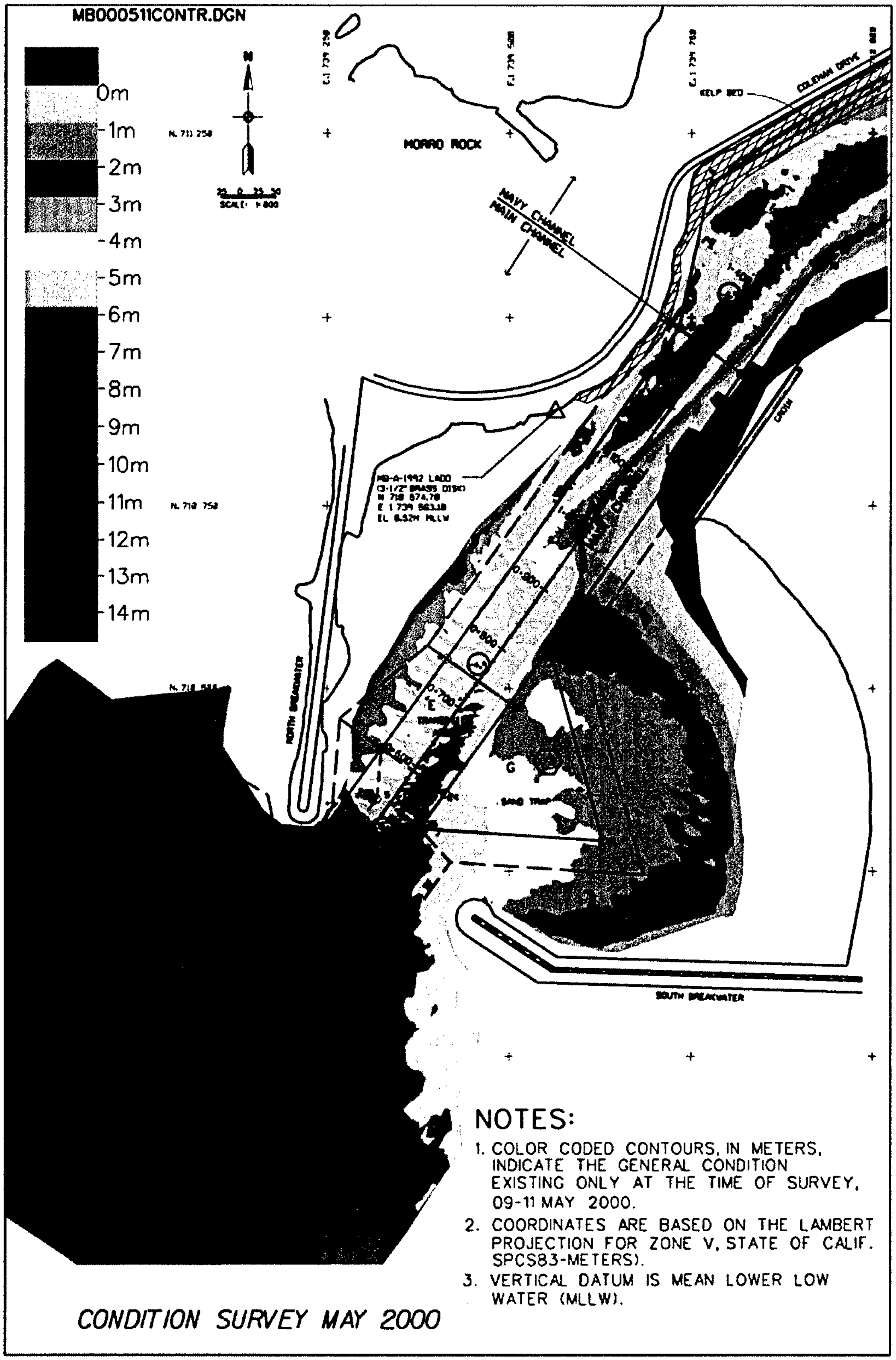


Figure A17. Bathymetry at Morro Bay entrance, 9-11 May 2000

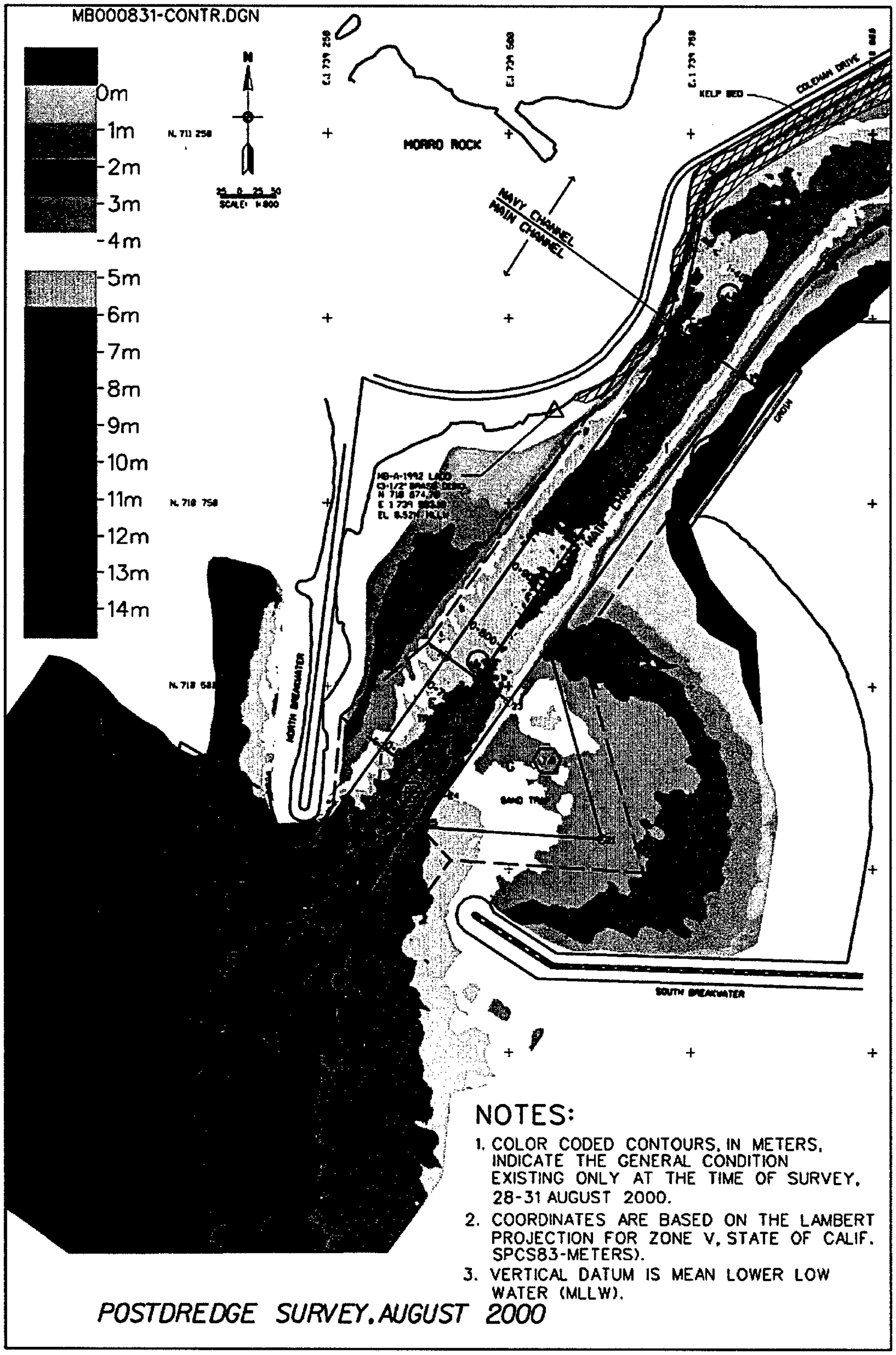


Figure A18. Bathymetry at Morro Bay entrance, 28-31 August 2000

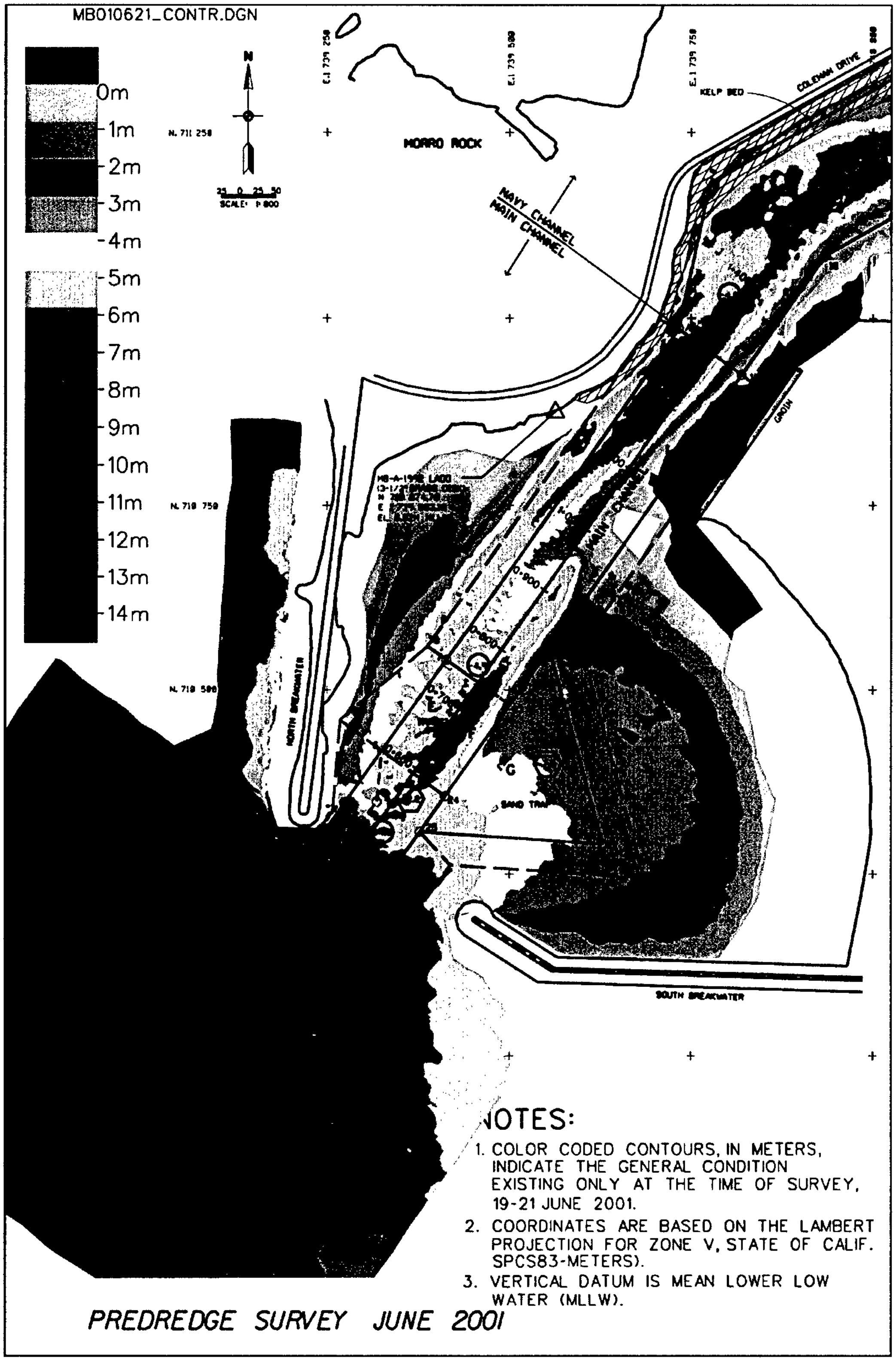


Figure A19. Bathymetry at Morro Bay entrance, 19-21 June 2001

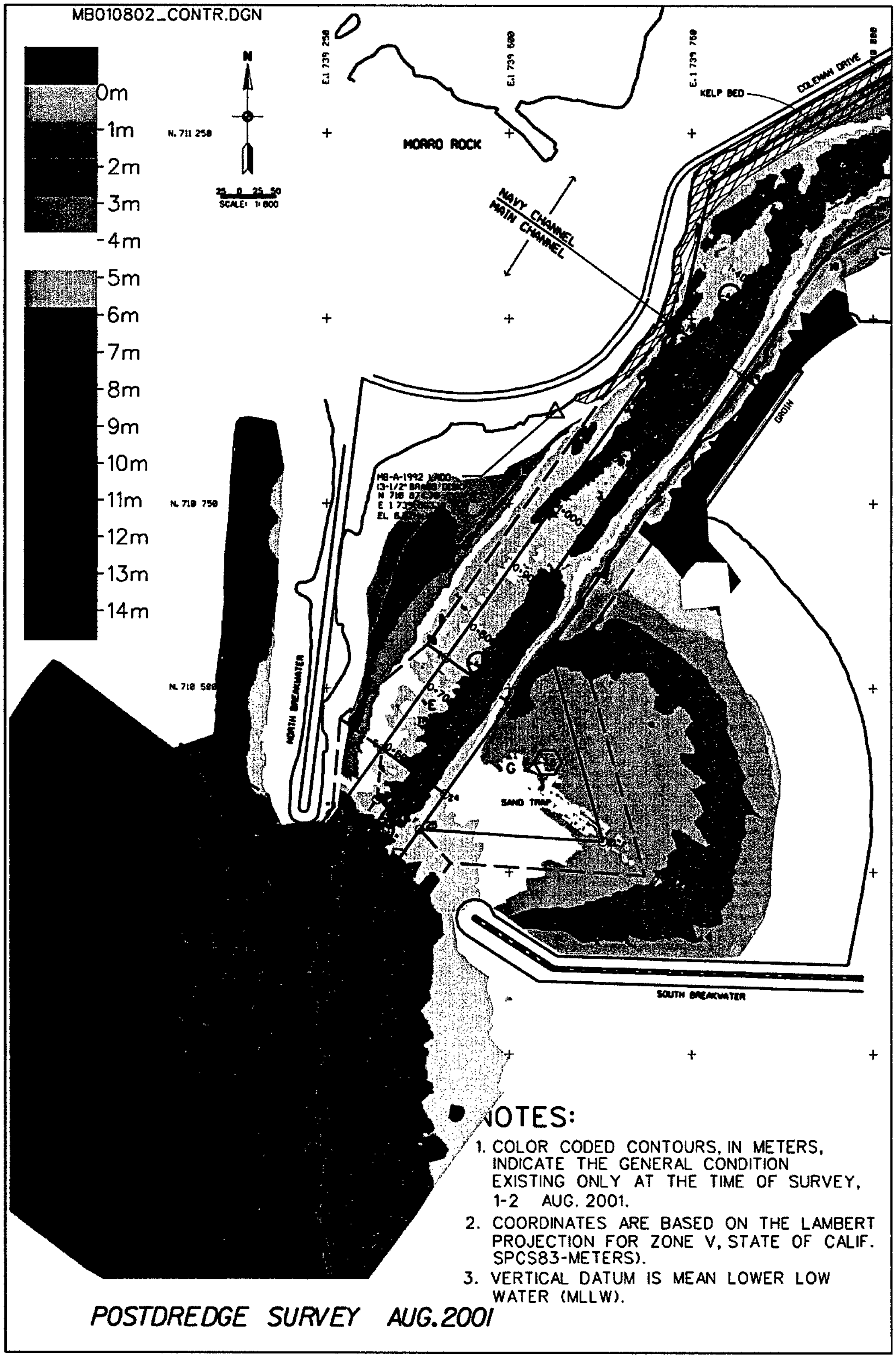


Figure A20. Bathymetry at Morro Bay entrance, 1-2 August 2001

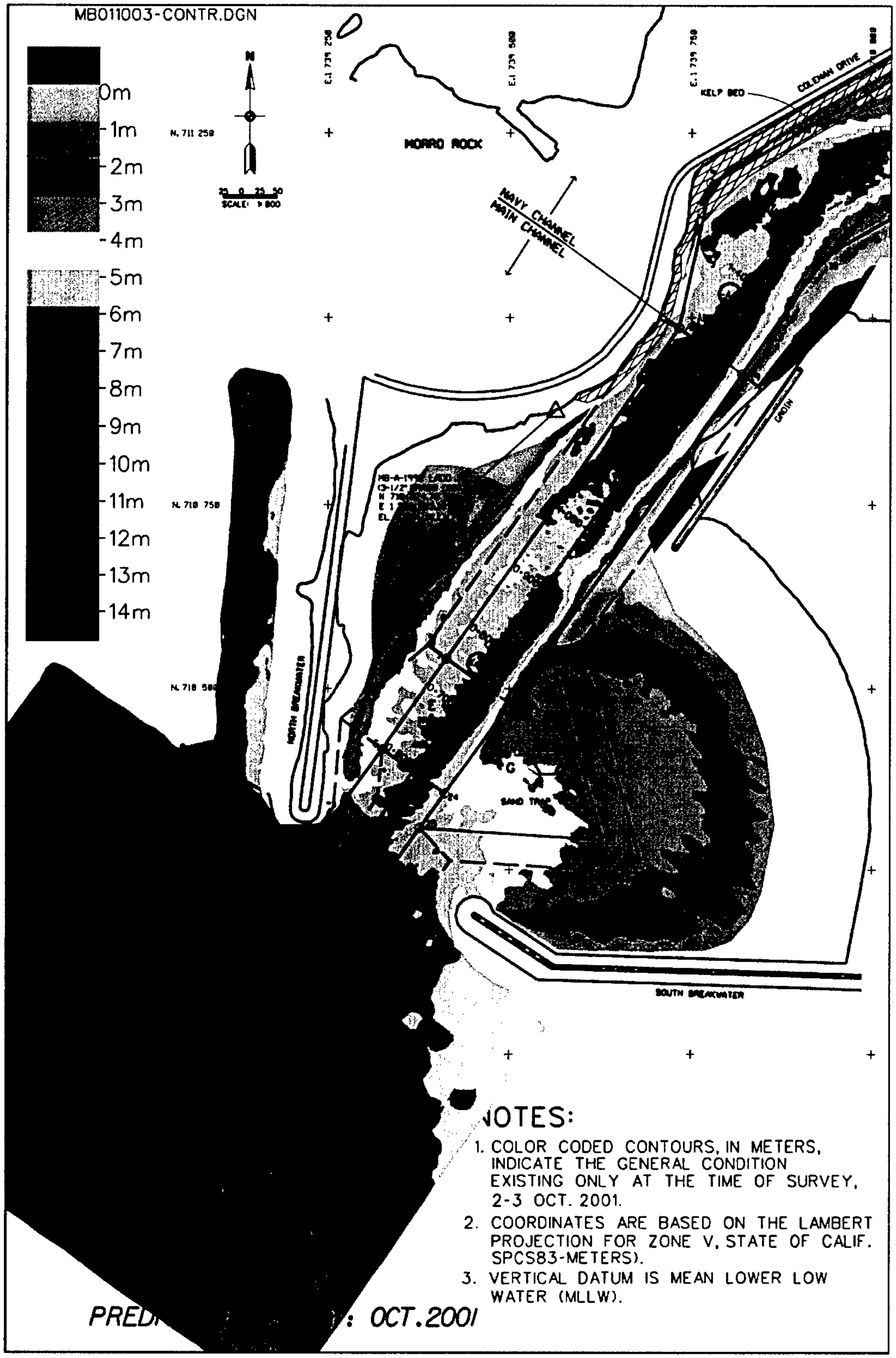


Figure A21. Bathymetry at Morro Bay entrance, 2-3 October 2001

Appendix B
Physical and Numerical Model Wave Comparison Plots

This appendix presents plots of wave height variation along the navigation channel center line, as estimated by physical and numerical modeling studies conducted during planning and design of the Morro Bay Harbor project. Distances along the channel center line are measured from a reference point seaward of the entrance. Physical model results were converted to amplification factors by dividing channel wave heights by corresponding incident wave height. Plots for representative incident waves in the preproject condition, called the existing condition in the original model studies, are presented first. These are followed by plots for the same incident waves in the with-project condition.

Physical and numerical model incident wave periods and directions were matched as nearly as possible from the original studies for comparison. The original HARBD numerical model studies represent regular (monochromatic) waves. Physical model results shown here are based on more realistic unidirectional irregular wave experiments. The project condition shown was Plan 14 in the original physical model studies and Alternative 6 in the numerical model studies.

The present numerical model for harbor waves, CGWAVE, was run for selected Morro Bay Harbor cases to help assess improvements in this analysis tool since the original numerical model studies were conducted. CGWAVE was run to duplicate conditions of physical model tests as nearly as possible, including unidirectional, irregular waves, similar bathymetry, and wave breaking.

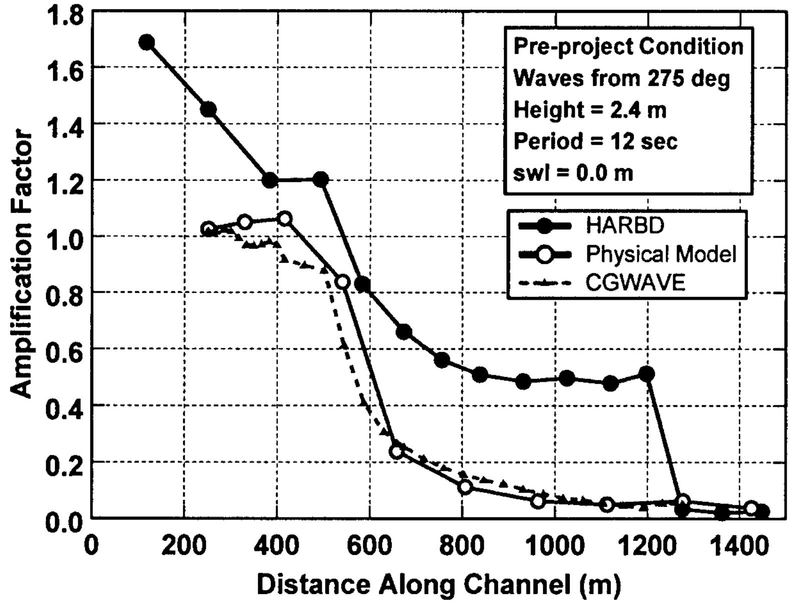
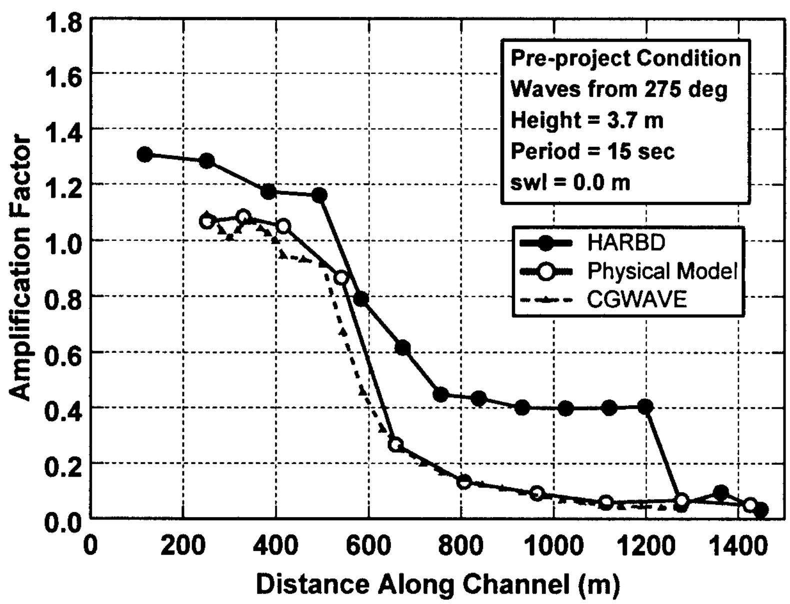


Figure B1.



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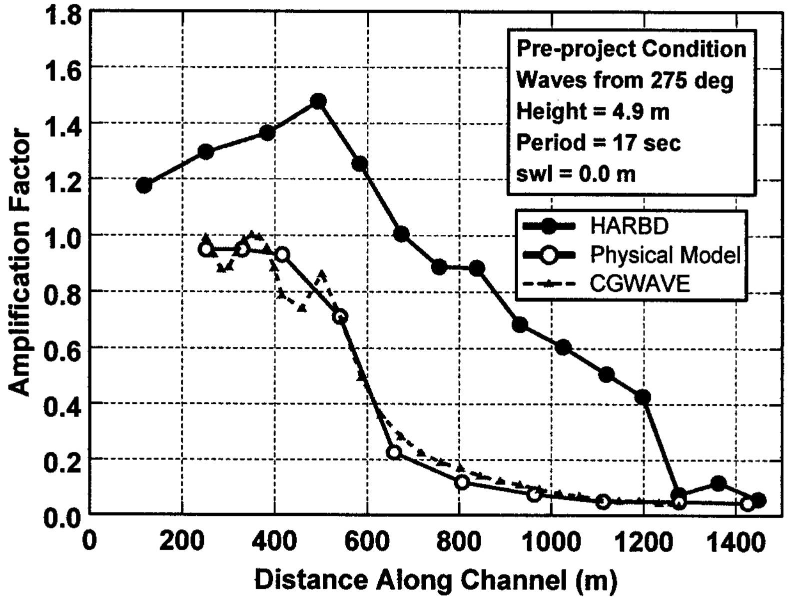
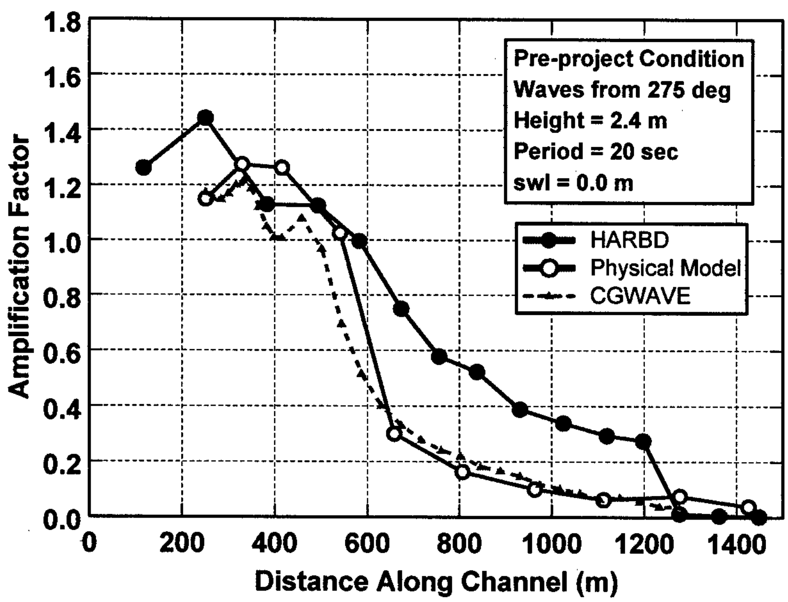


Figure B3.



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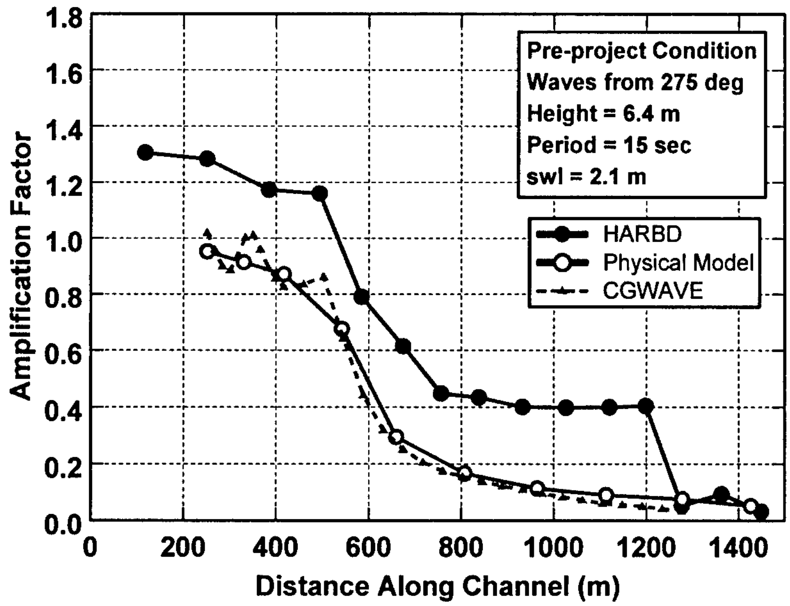

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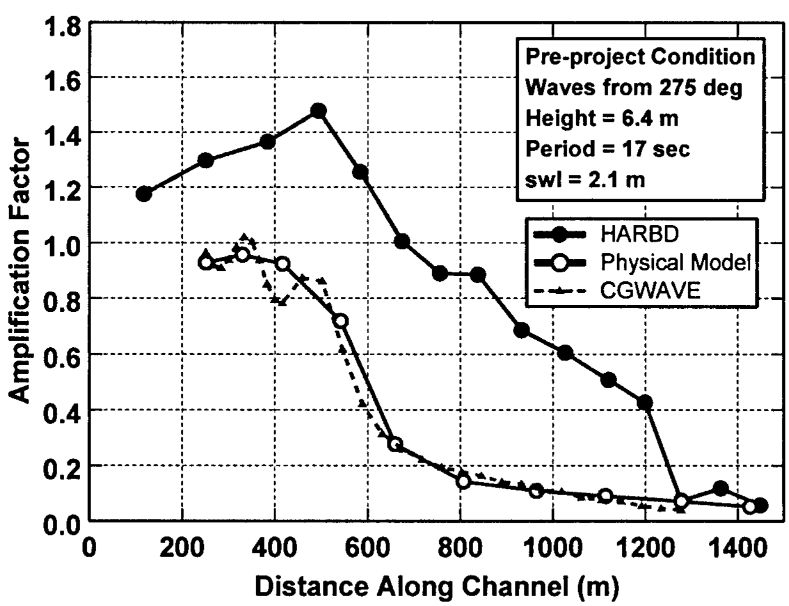

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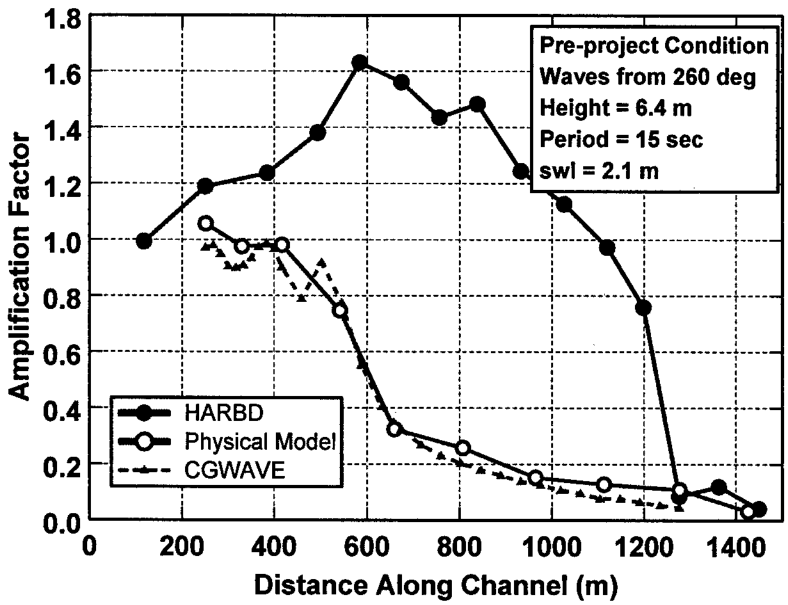

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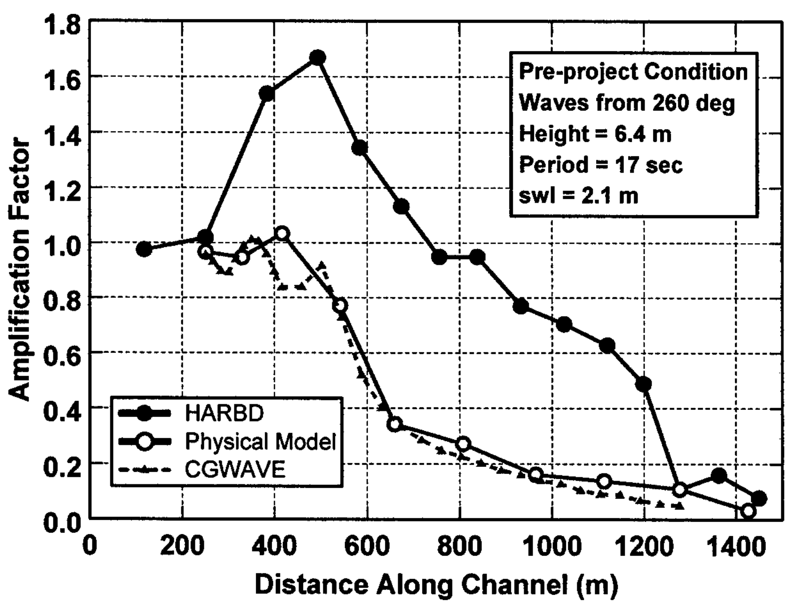

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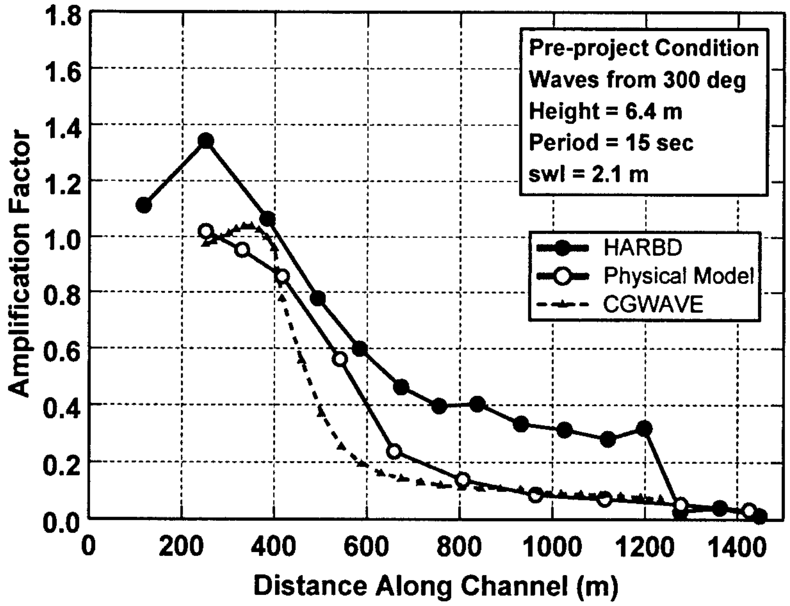

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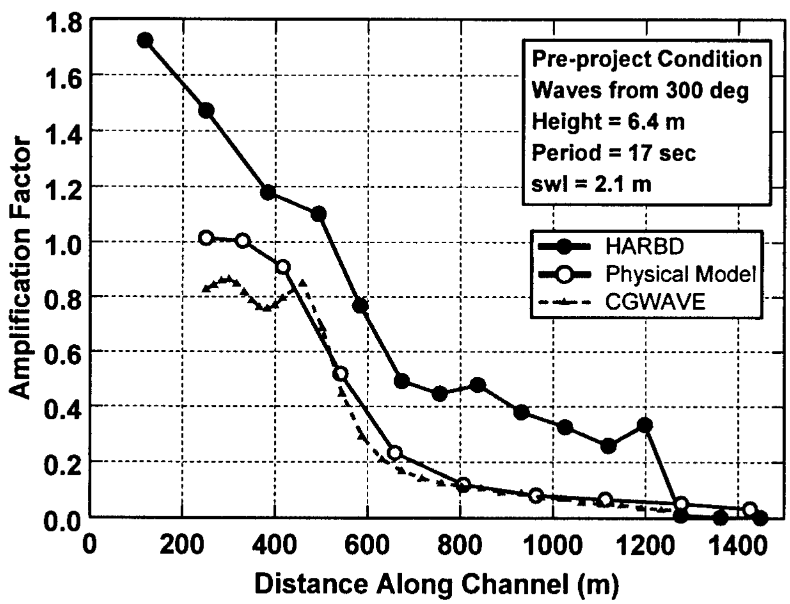

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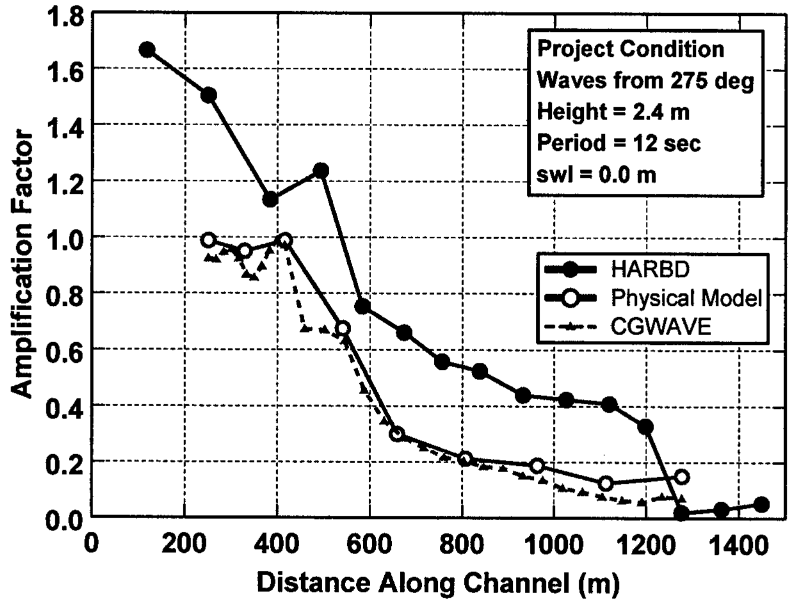

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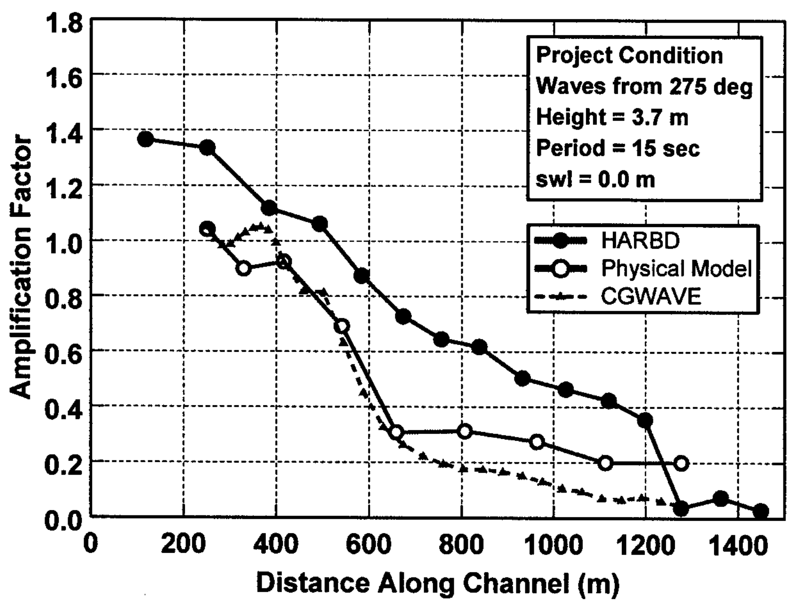
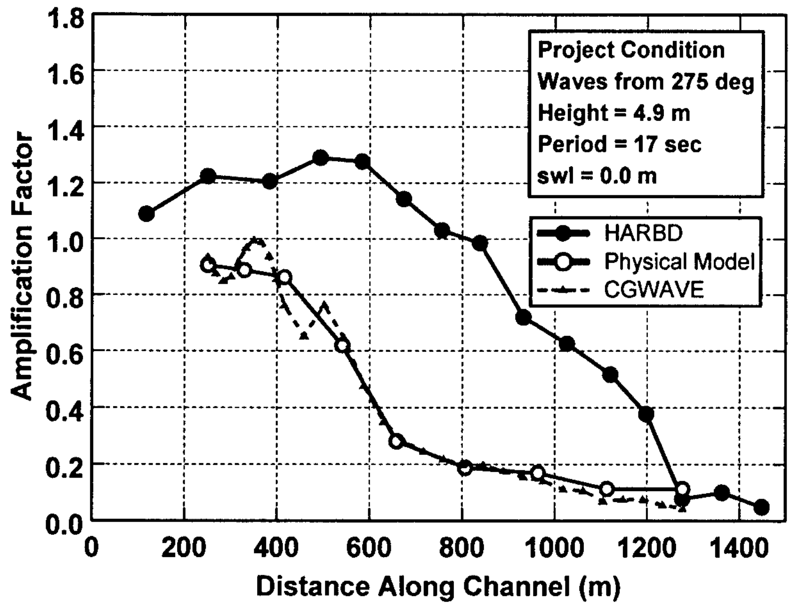


Figure B12.



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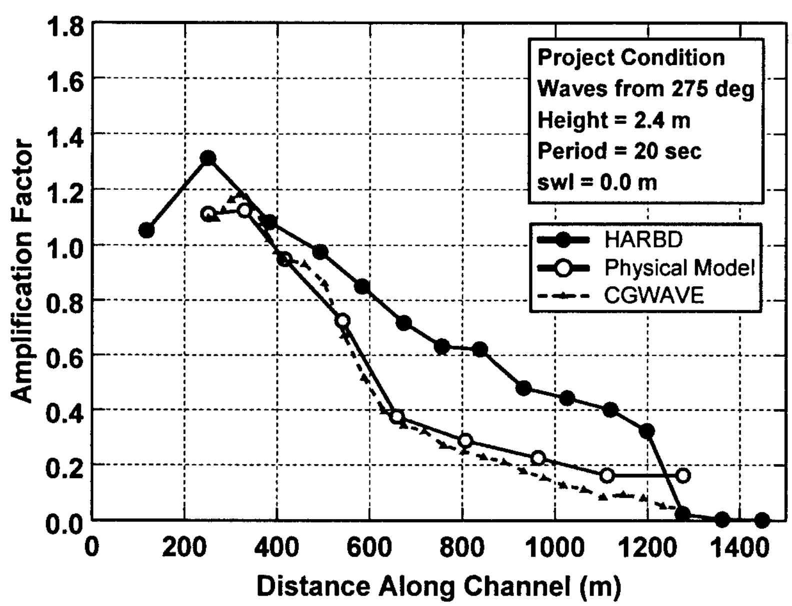

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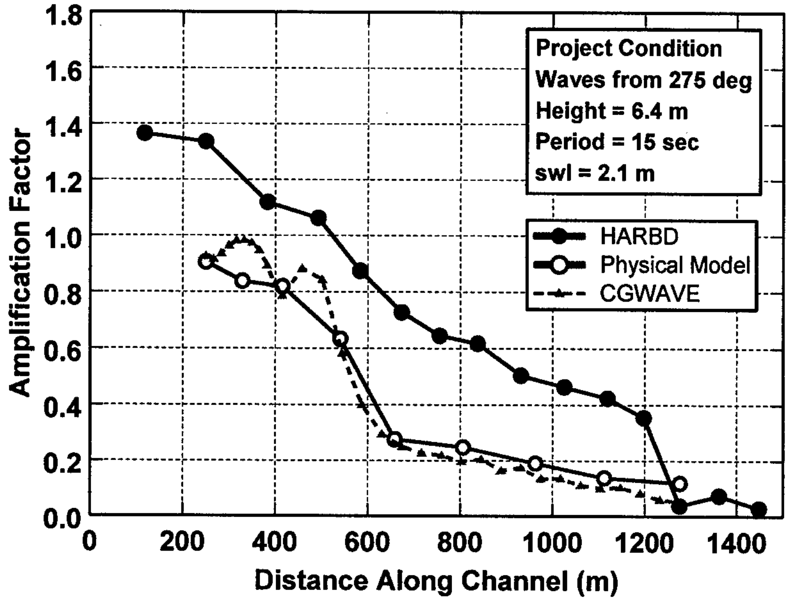

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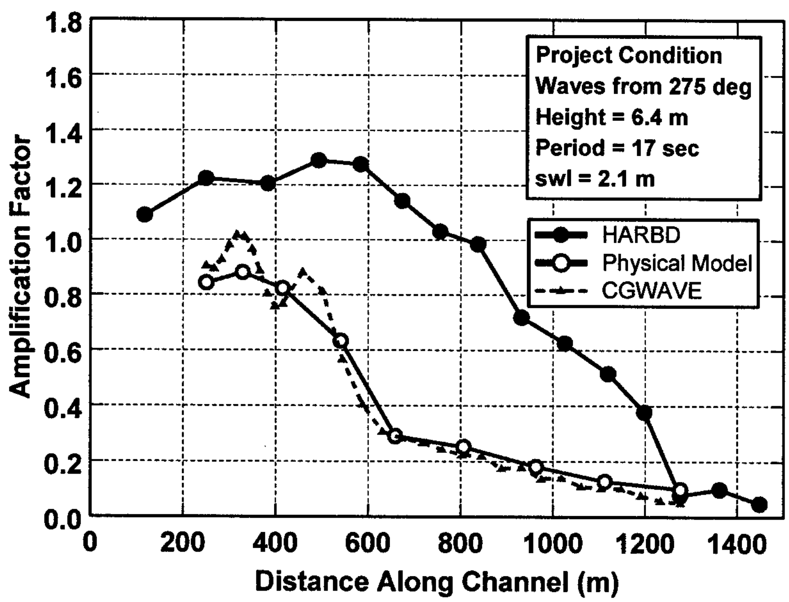

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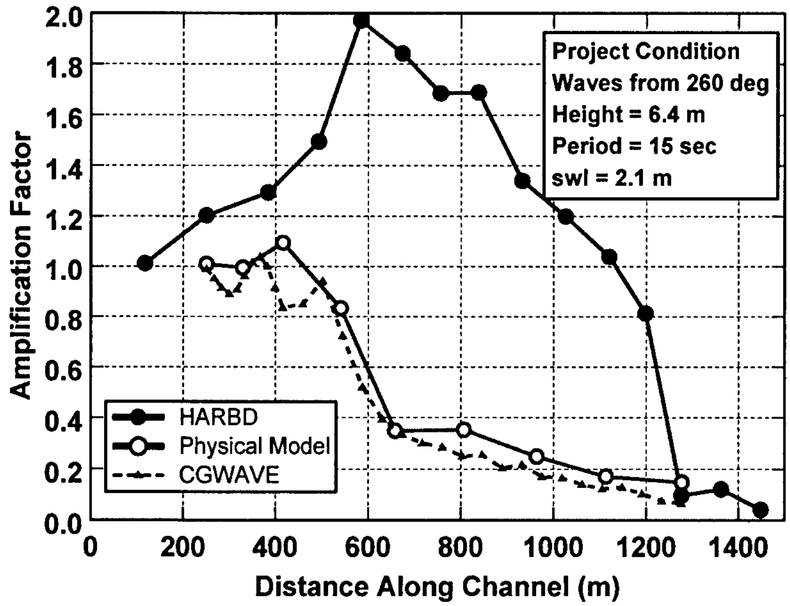

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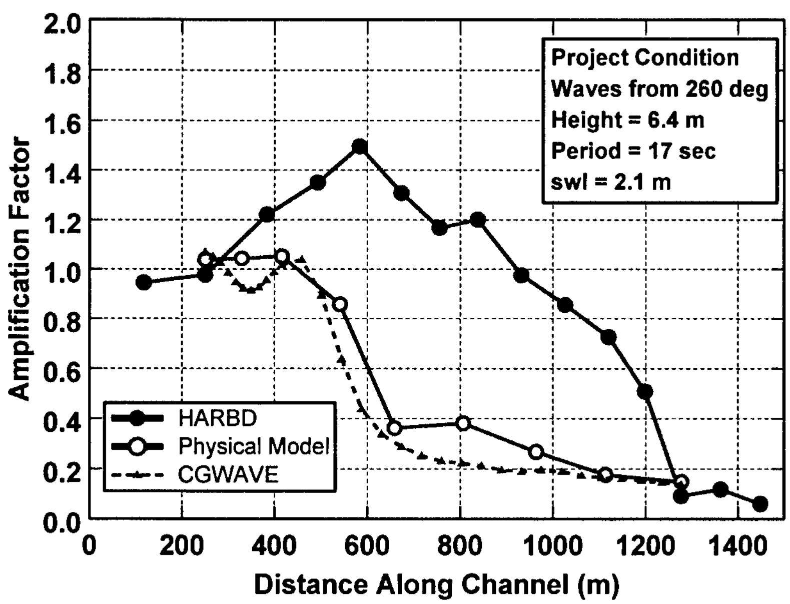

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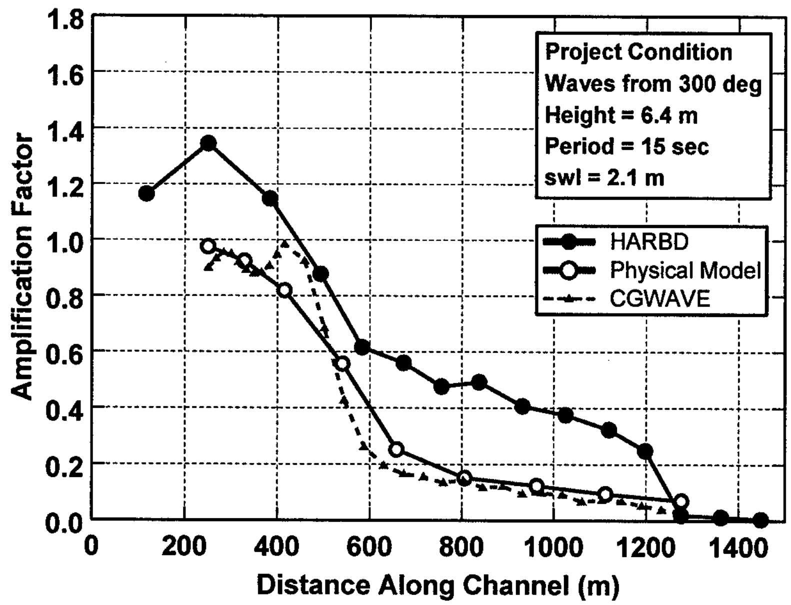

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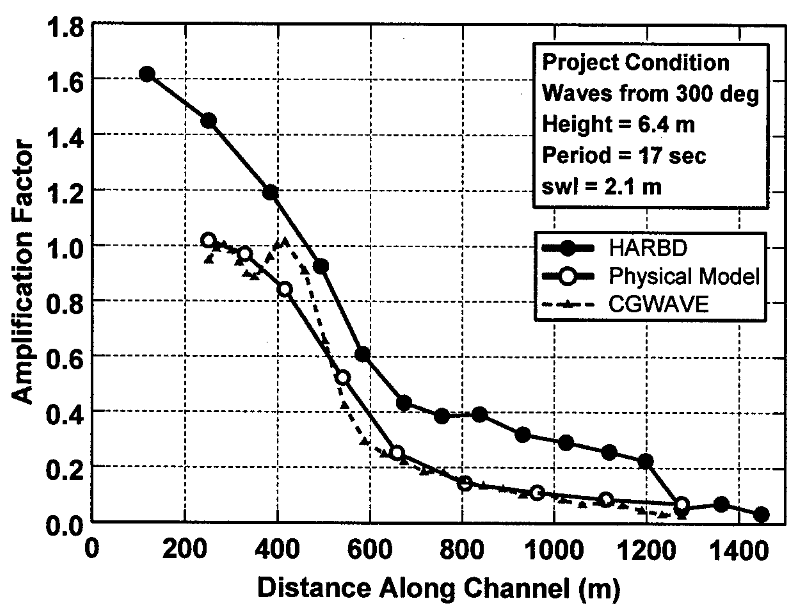

Figure B20.

Appendix C
Bathymetric Survey Difference Contour Plots

This appendix presents contour plots of depth differences between successive bathymetric surveys of Morro Bay entrance taken between April 1996 and May 2000. Survey intervals during which entrance and outer harbor dredging occurred are not included. Depth differences are in meters. Positive differences indicate shoaling over the survey interval. Negative differences indicate erosion over the survey interval.

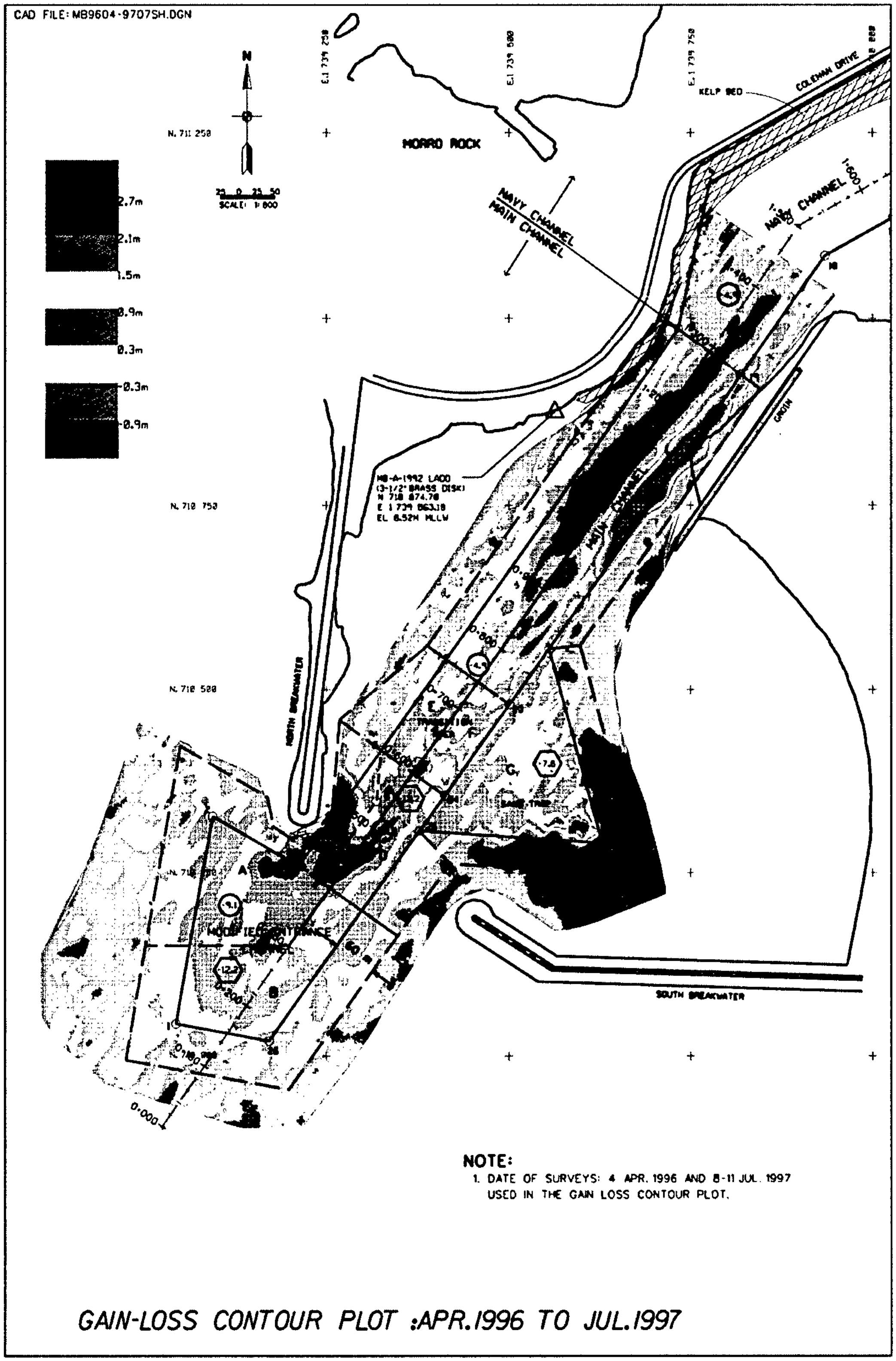

Figure C1. Bathymetry changes at Morro Bay entrance between 4 April 1996 and 8-11 July 1997

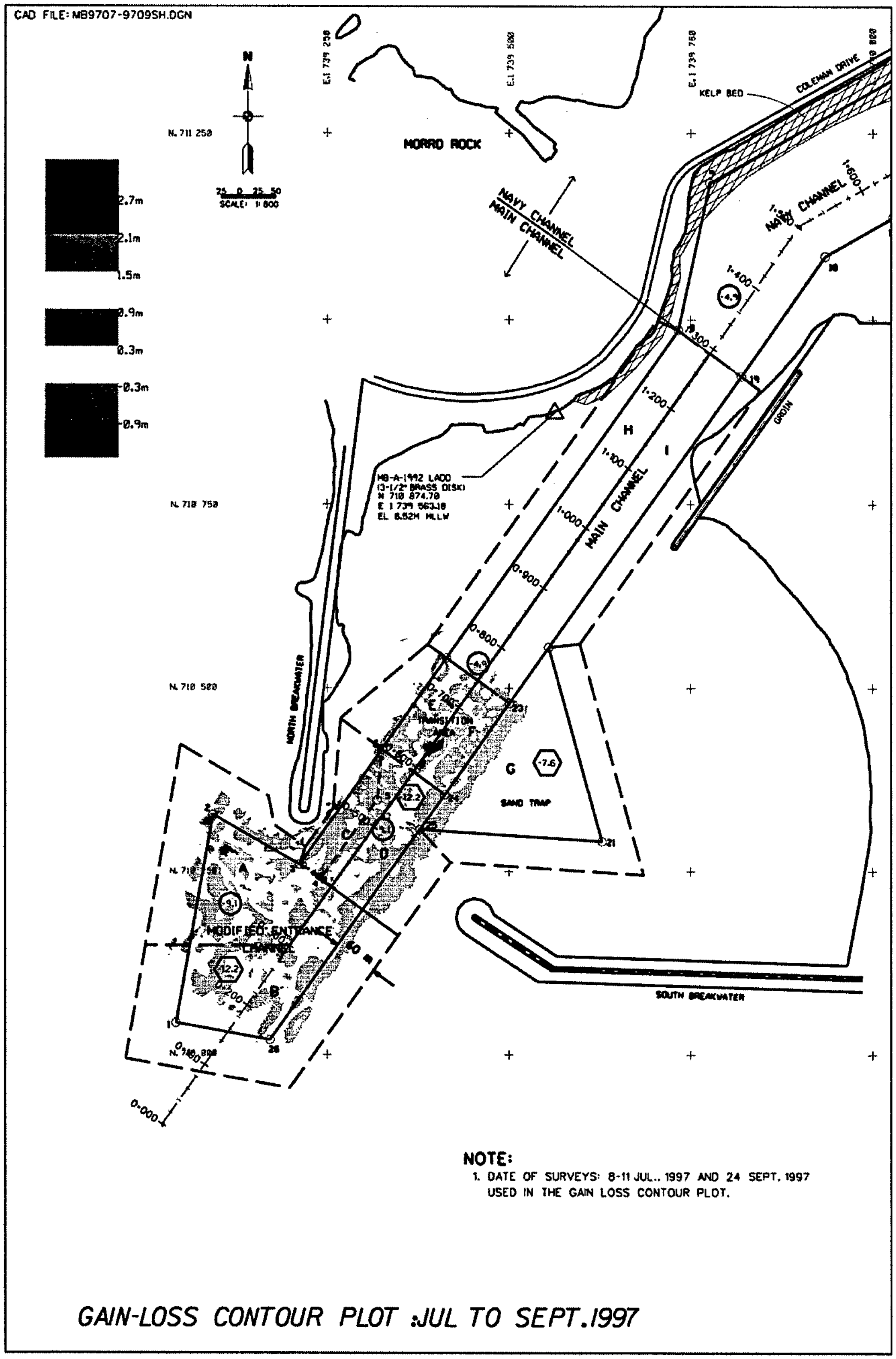

Figure C2. Bathymetry changes at Morro Bay entrance between 8-11 July 1997 and 24 September 1997

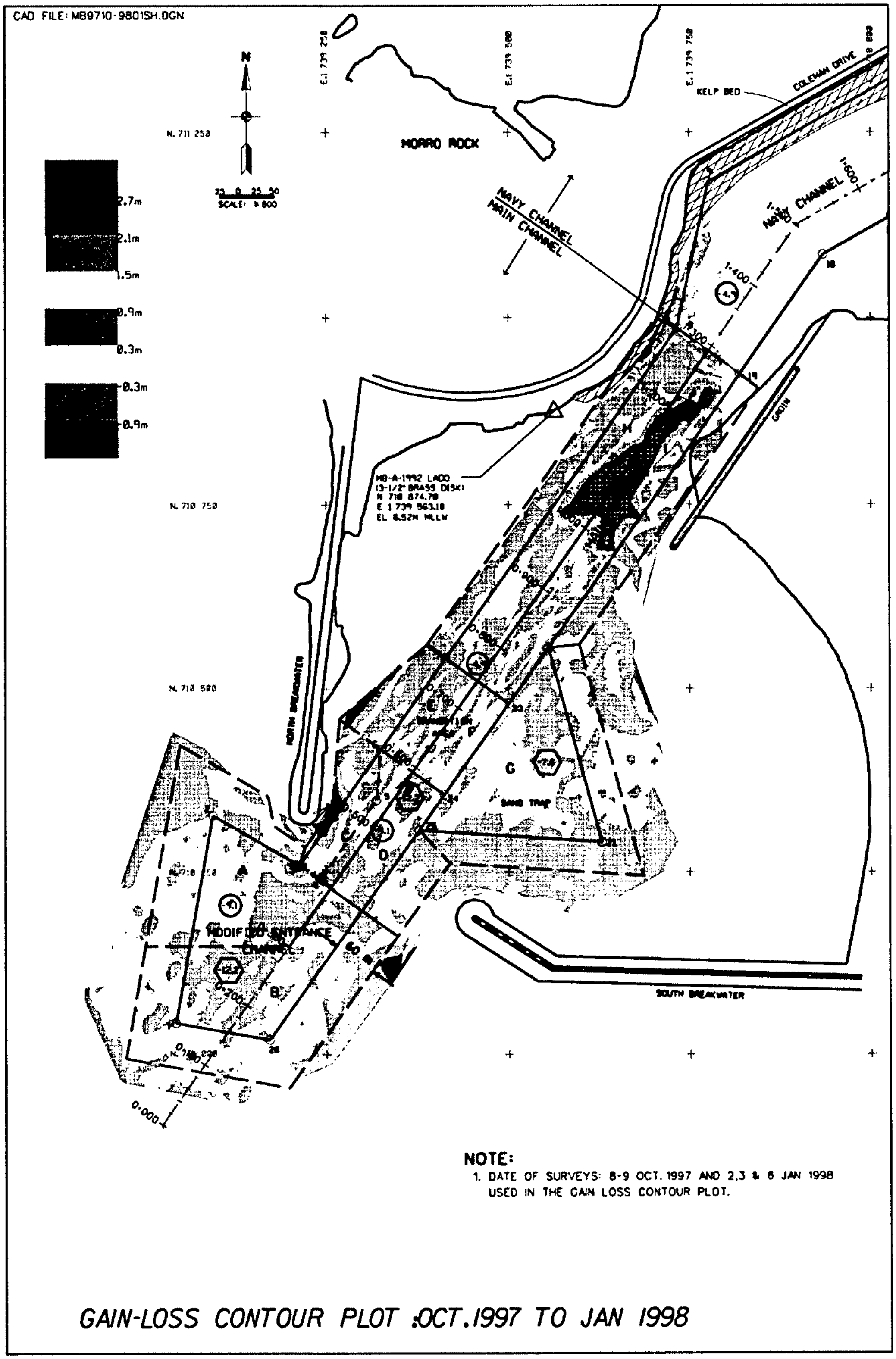

Figure C3. Bathymetry changes at Morro Bay entrance between 8-9 October 1997 and 2-6 January 1998

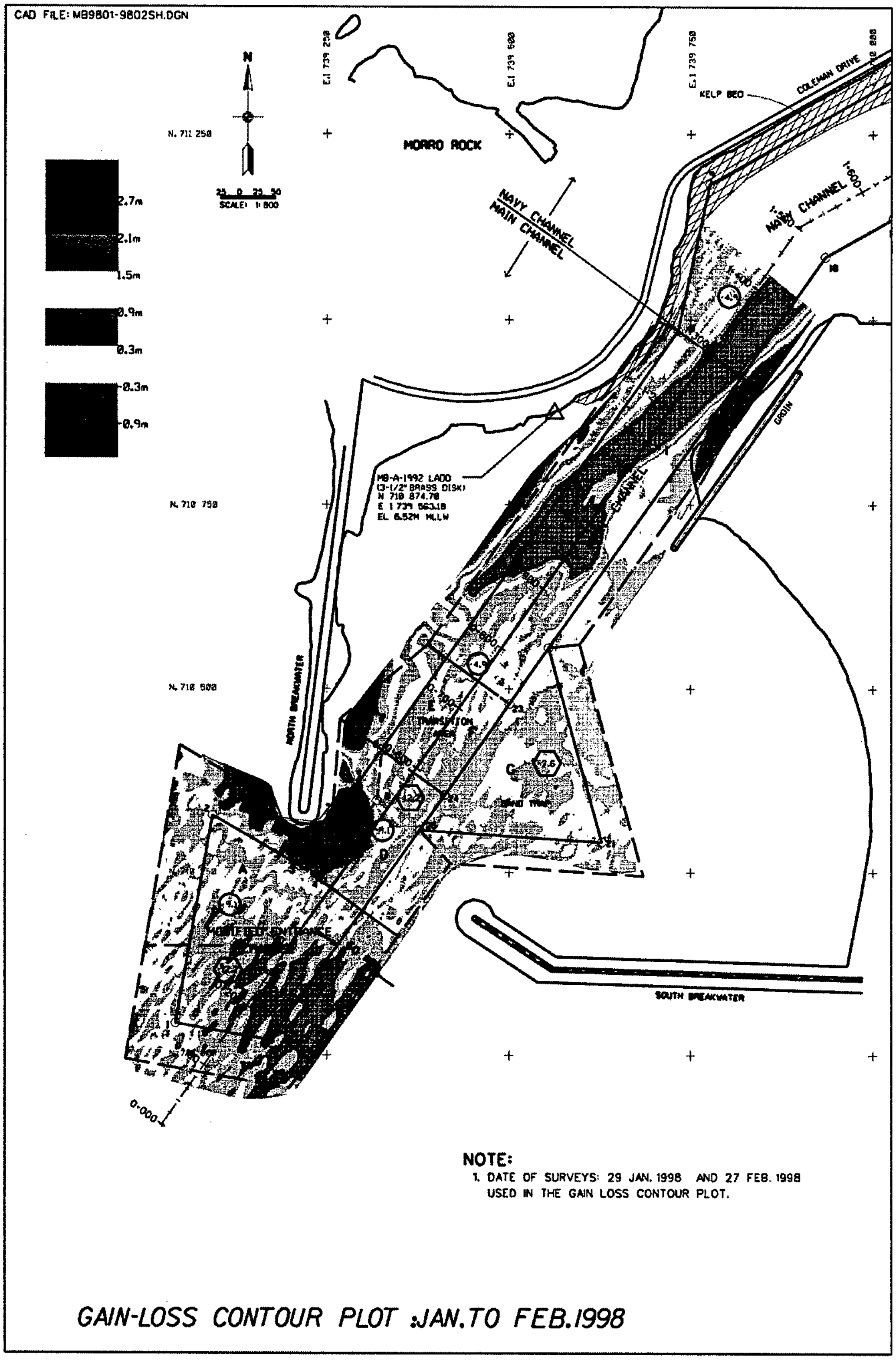

Figure C4. Bathymetry changes at Morro Bay entrance between 29 January 1998 and 27 February 1998

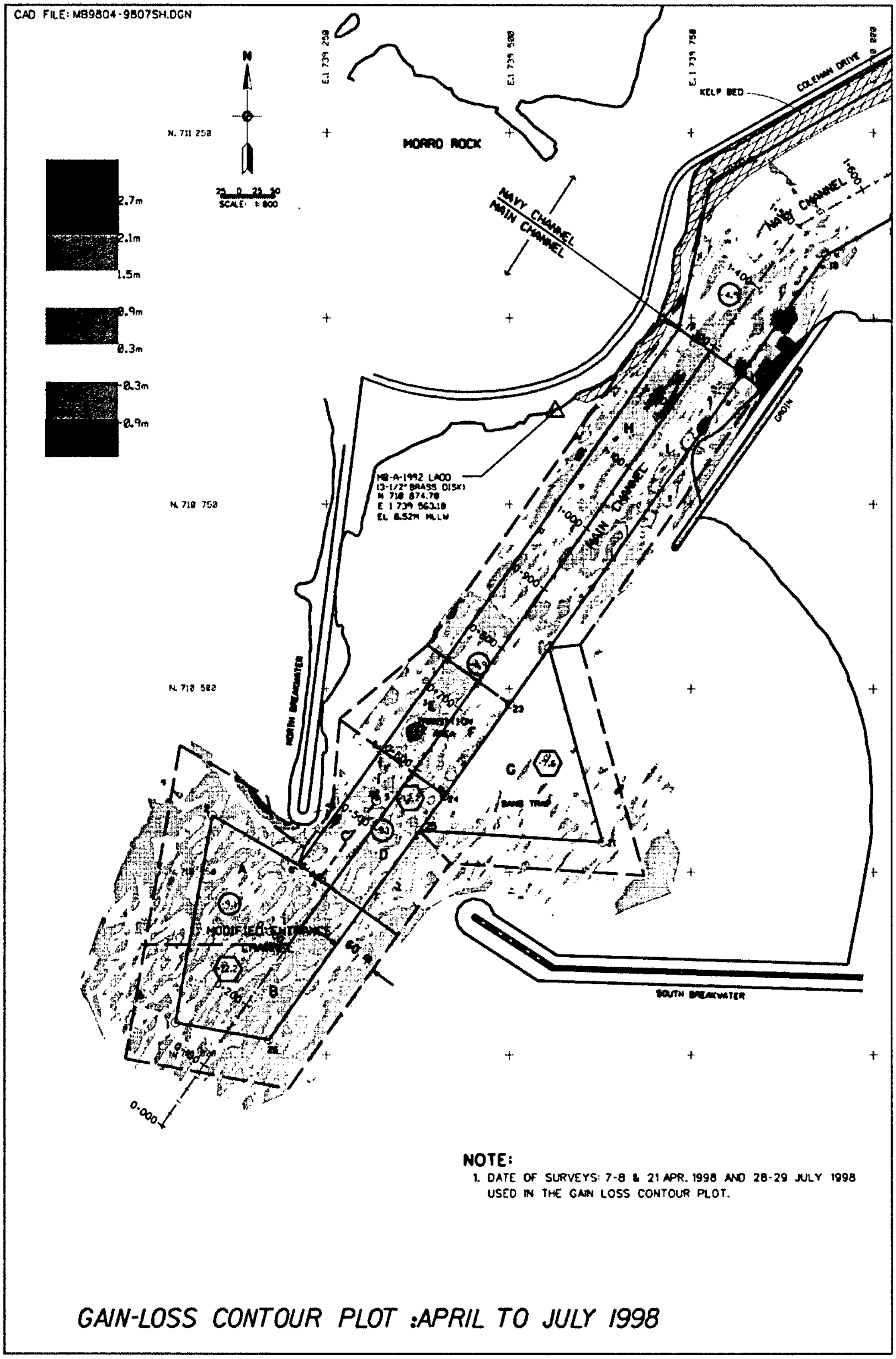

Figure C5. Bathymetry changes at Morro Bay entrance between 7-21 April 1998 and 28-29 July 1998

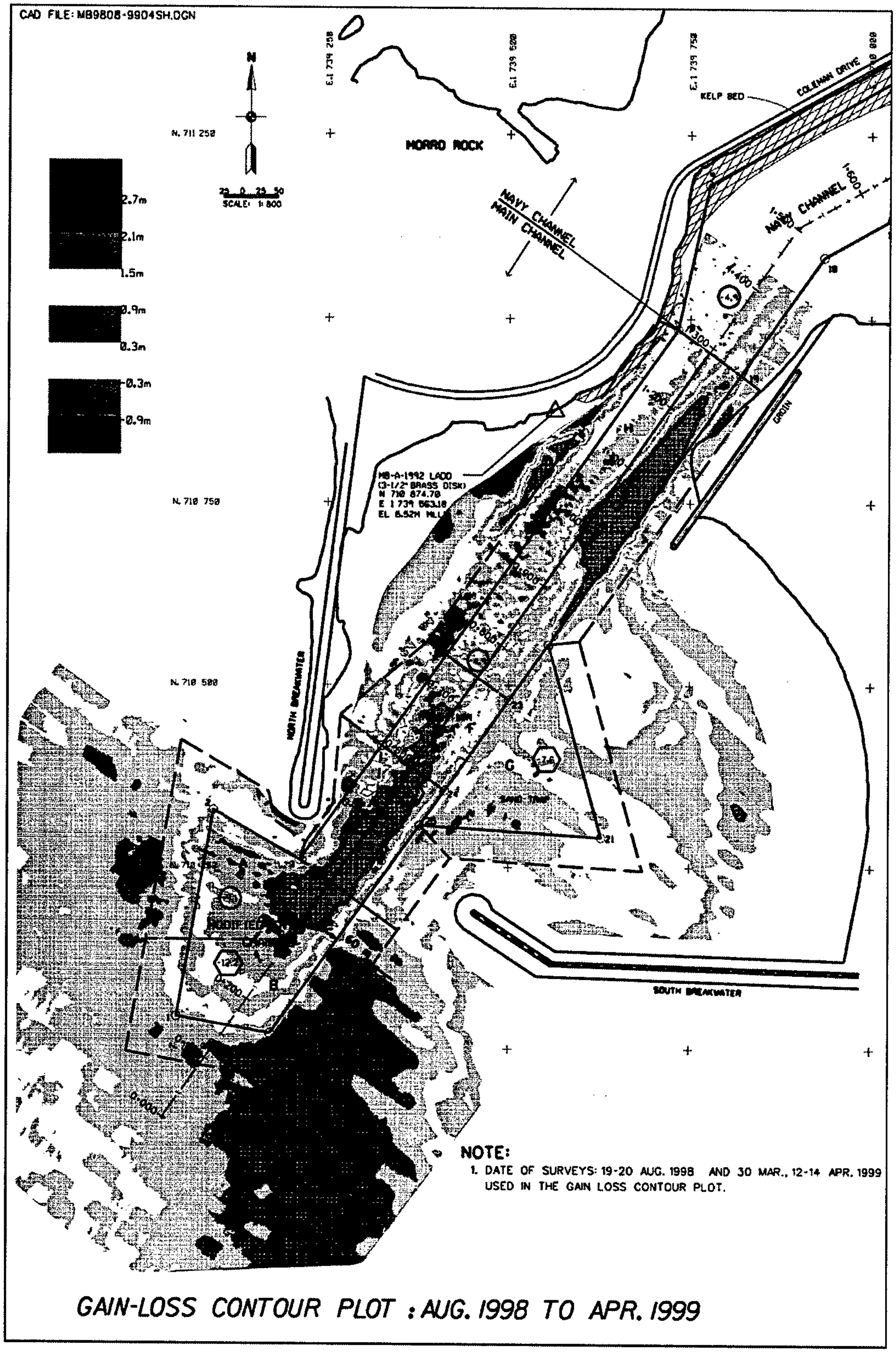

Figure C6. Bathymetry changes at Morro Bay entrance between 19-20 August 1998 and 30 March-14 April 1999

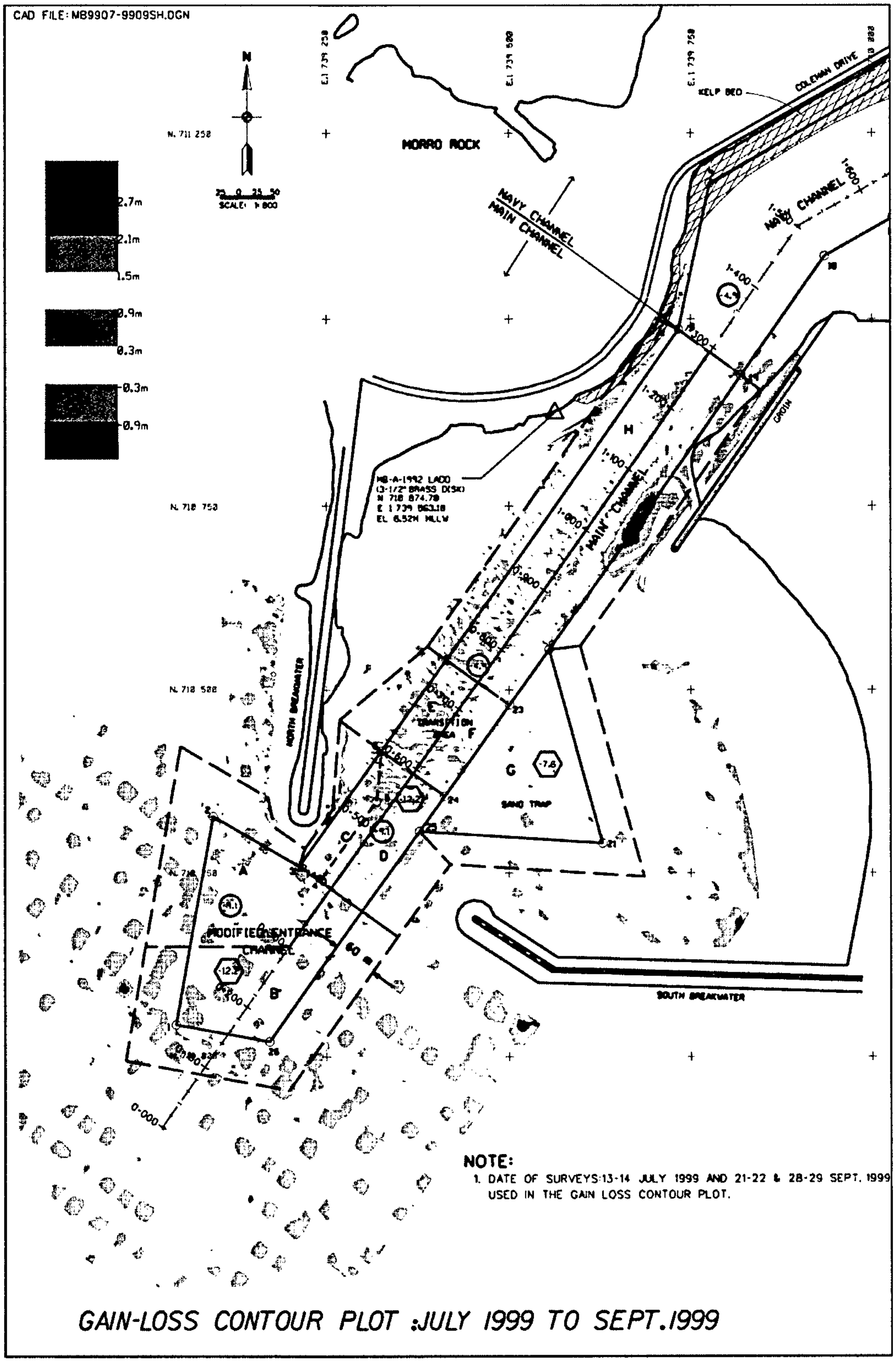

Figure C7. Bathymetry changes at Morro Bay entrance between 13-14 July 1999 and 21-29 September 1999

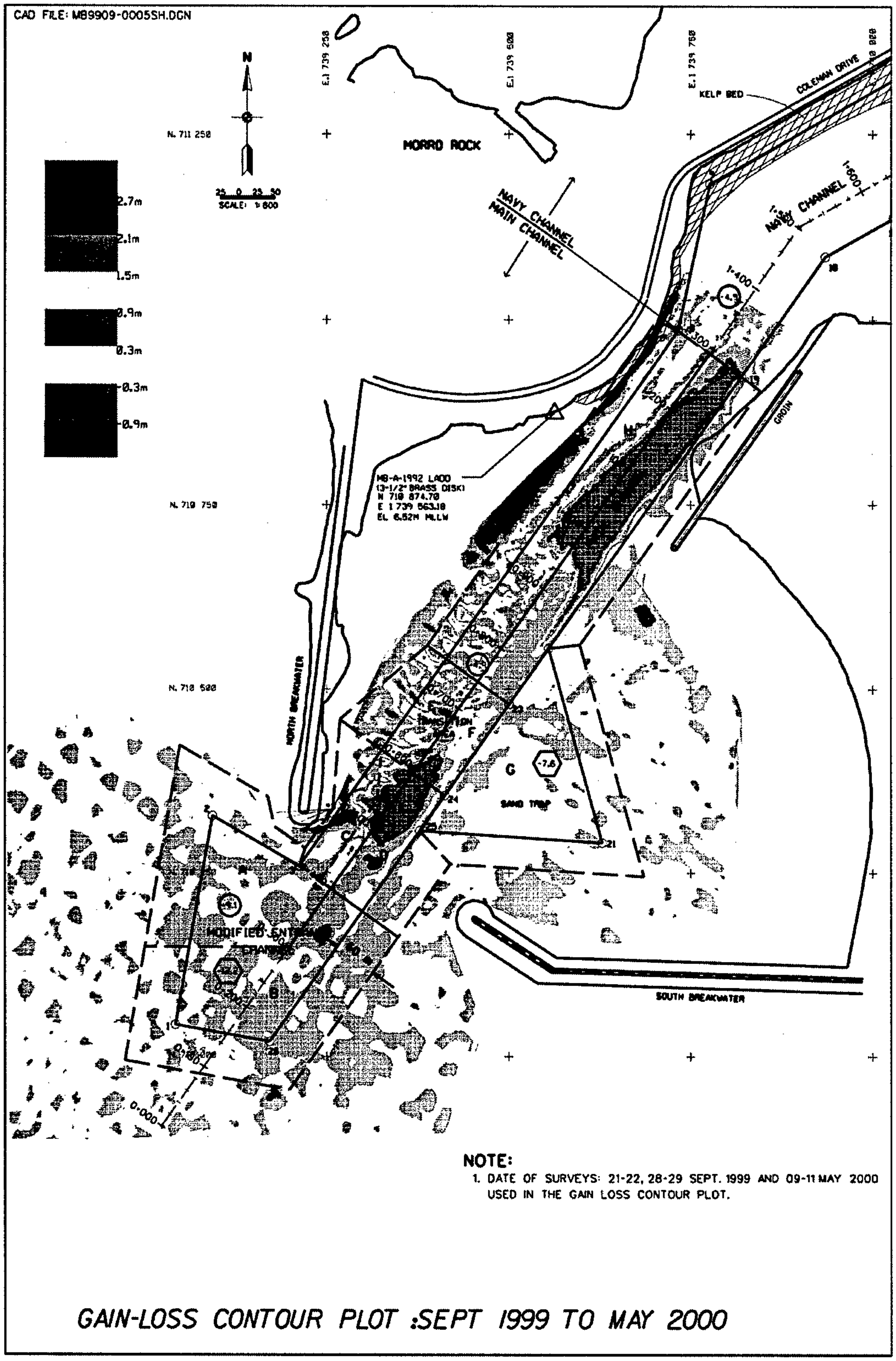

Figure C8. Bathymetry changes at Morro Bay entrance between 21-29 September 1999 and 9-11 May 2000

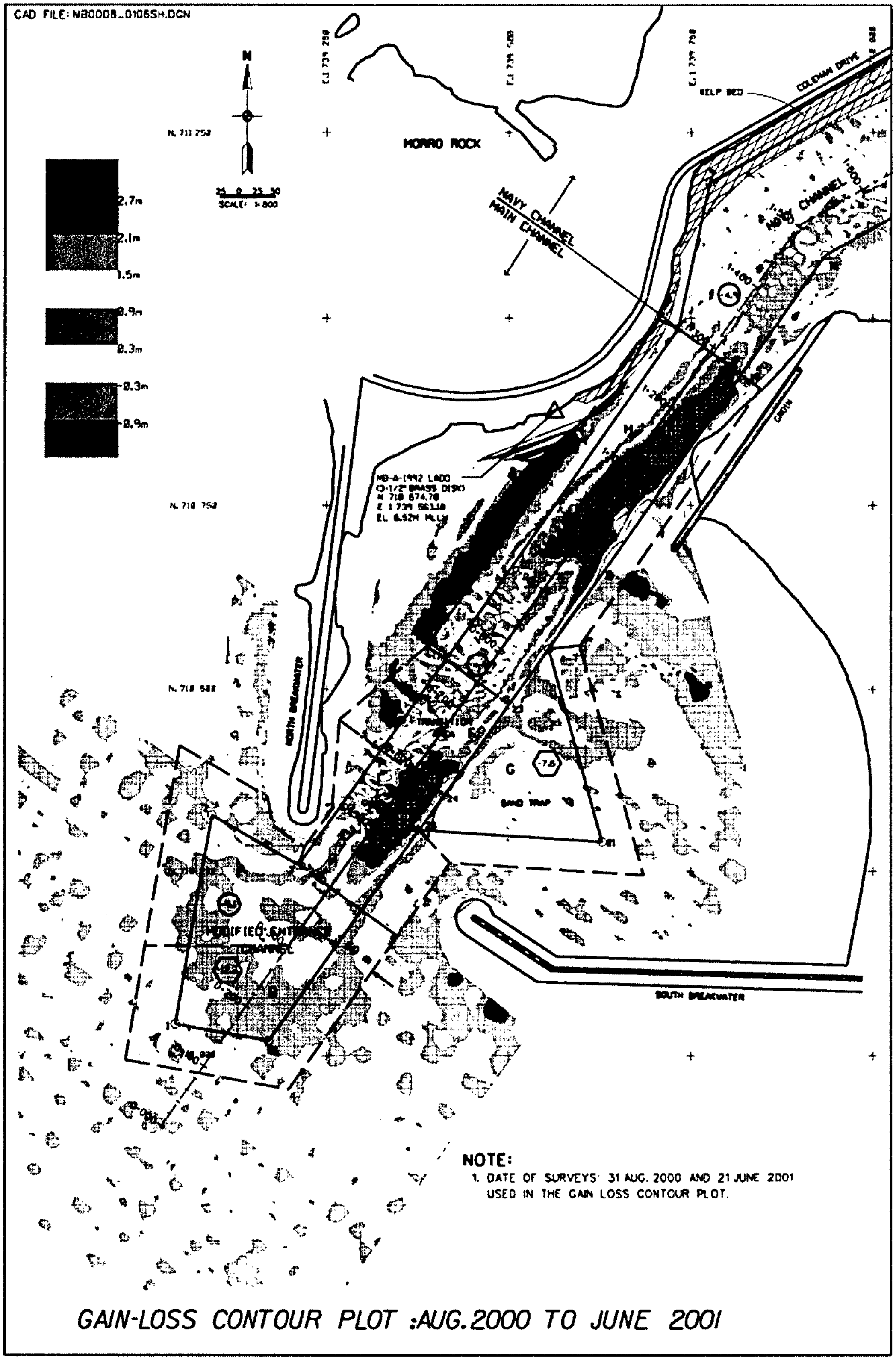

Figure C9. Bathymetry changes at Morro Bay entrance between 28-31 August 2000 and 19-21 June 2001

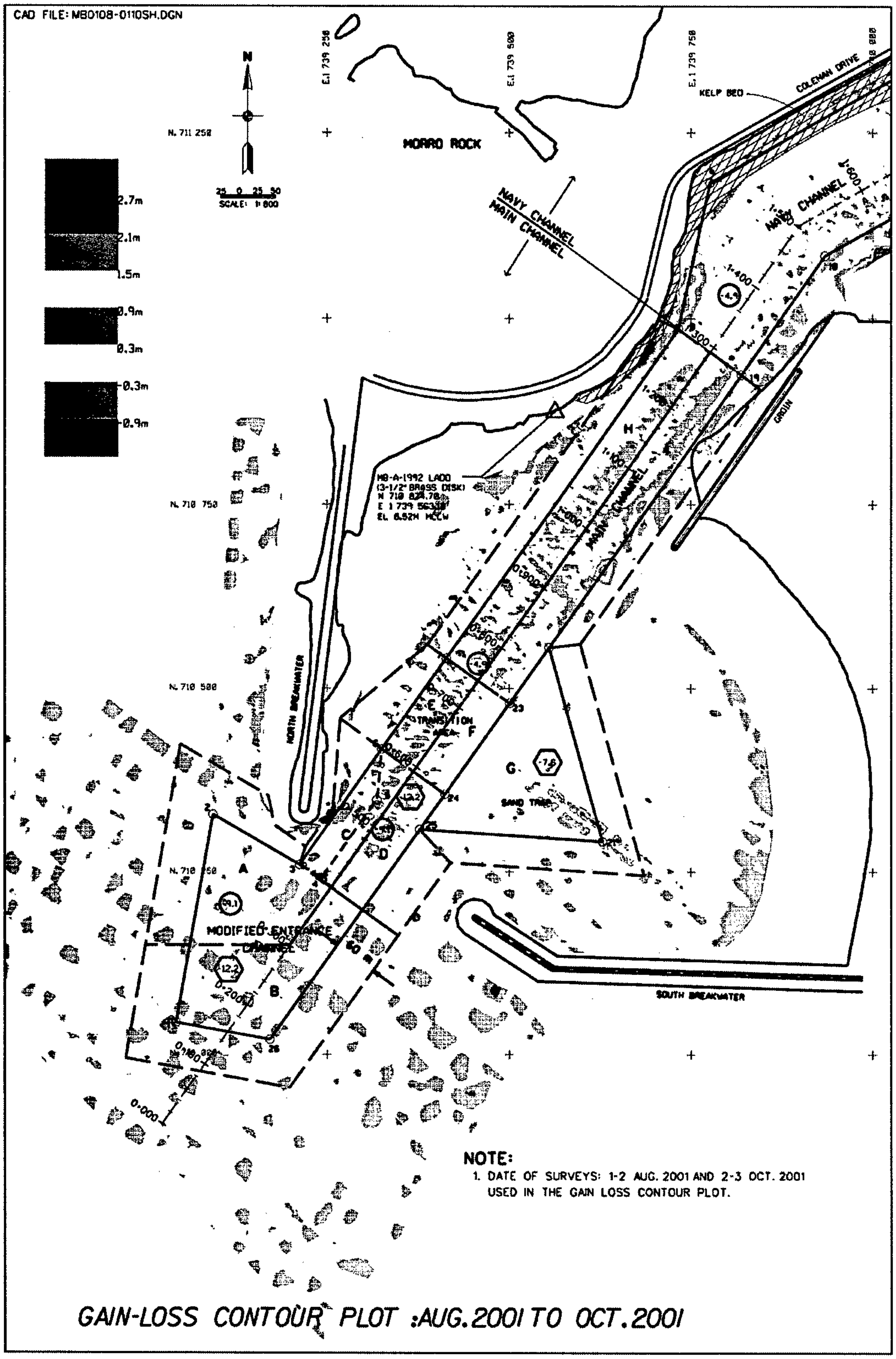

Figure C10. Bathymetry changes at Morro Bay entrance between 1-2 August 2001 and 2-3 October 2001

Appendix D
Locations of Targets

This appendix presents locations of targets established on the Morro Bay south breakwater. These targets were surveyed and used for control of the photogrammetric flights. Thirty targets were established for the June 1998 survey and 20 for the July 2000 survey. Seventeen of the 1998 targets were recovered for the 2000 survey. The scale of the maps is 2.54 cm = 6.1 m (1 in = 20 ft).

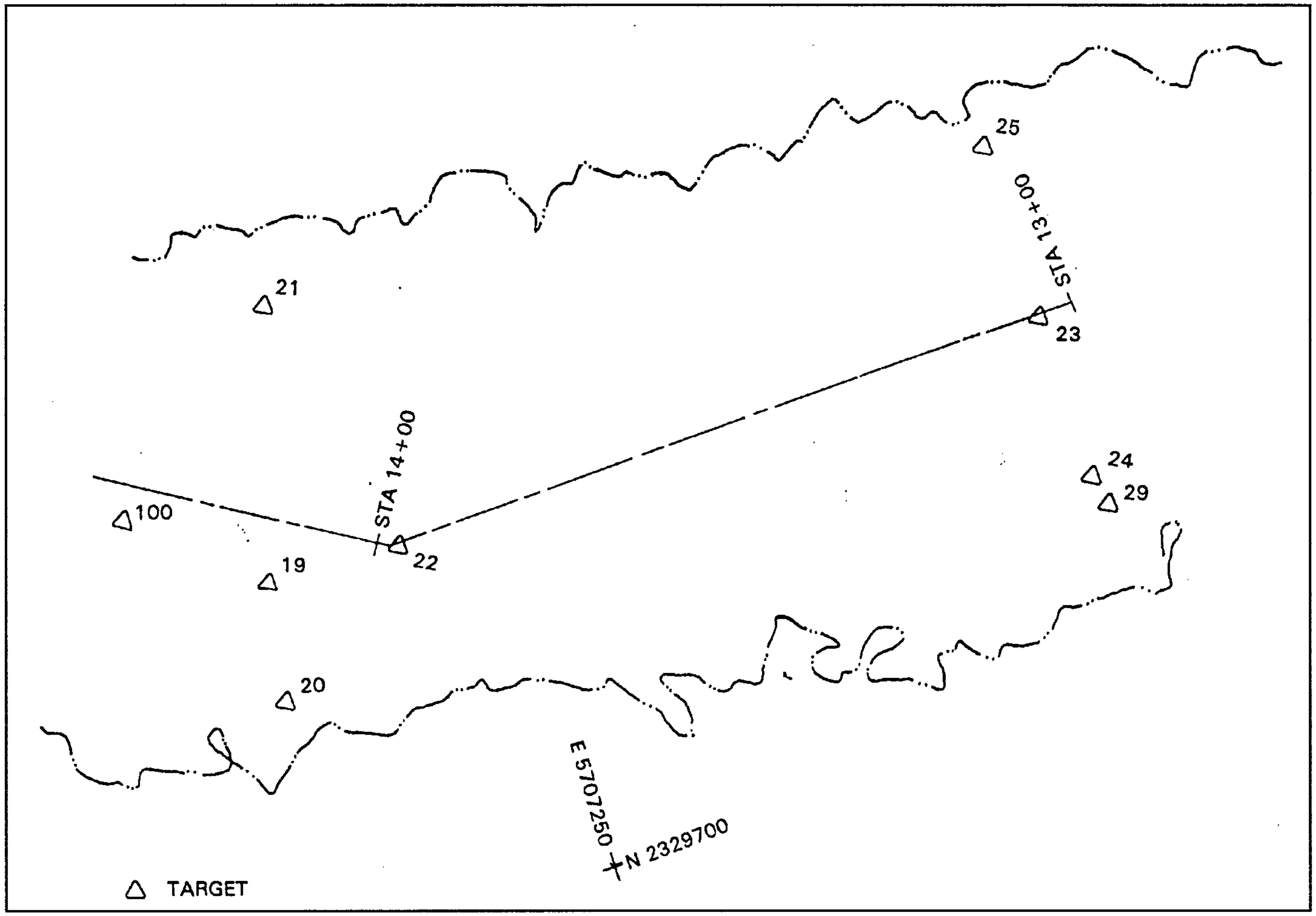

Figure D1. Locations of targets established on south breakwater, sta 13+00 – 14+40

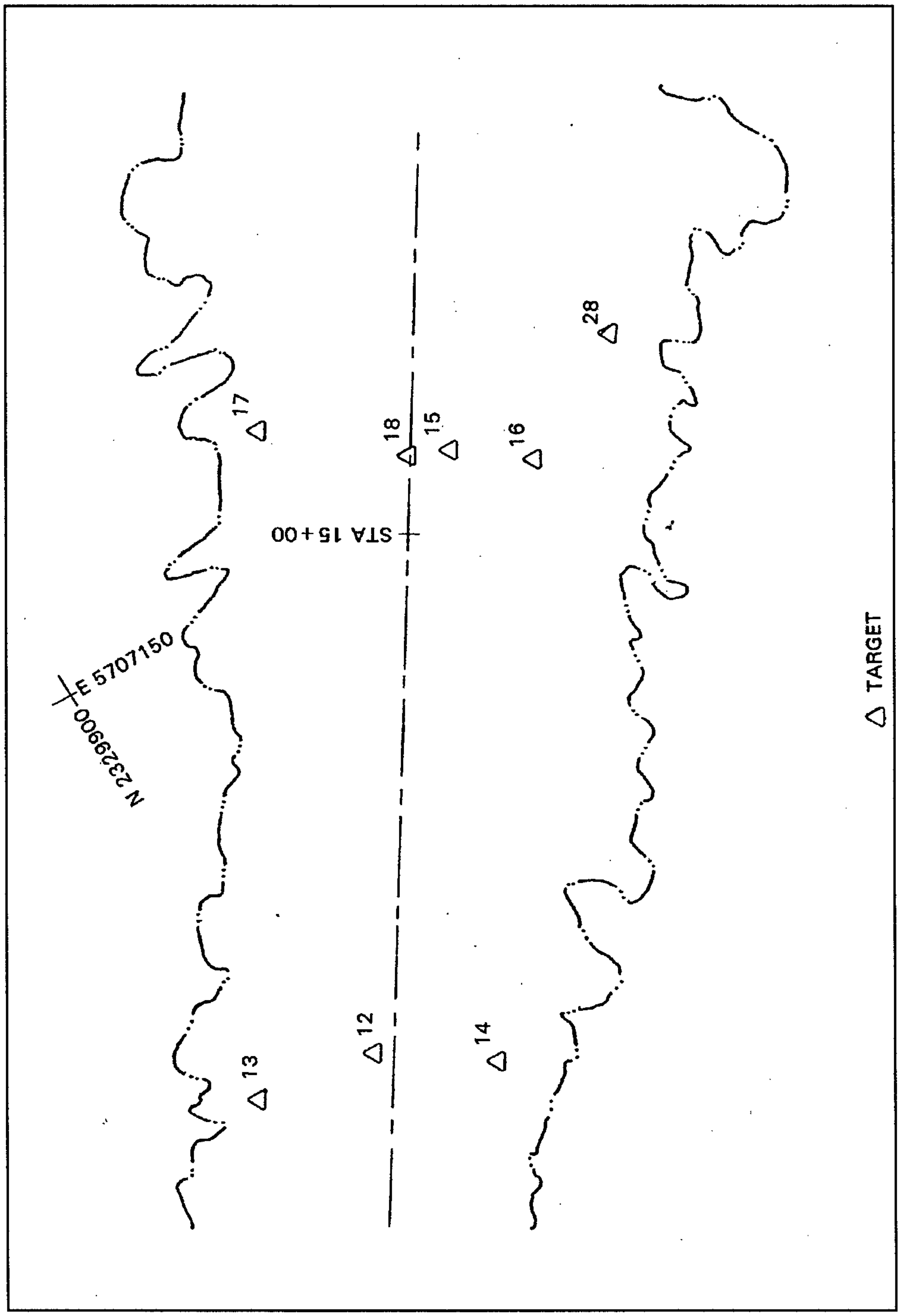

Figure D2. Locations of targets established on south breakwater, sta 14+40 – 15+94

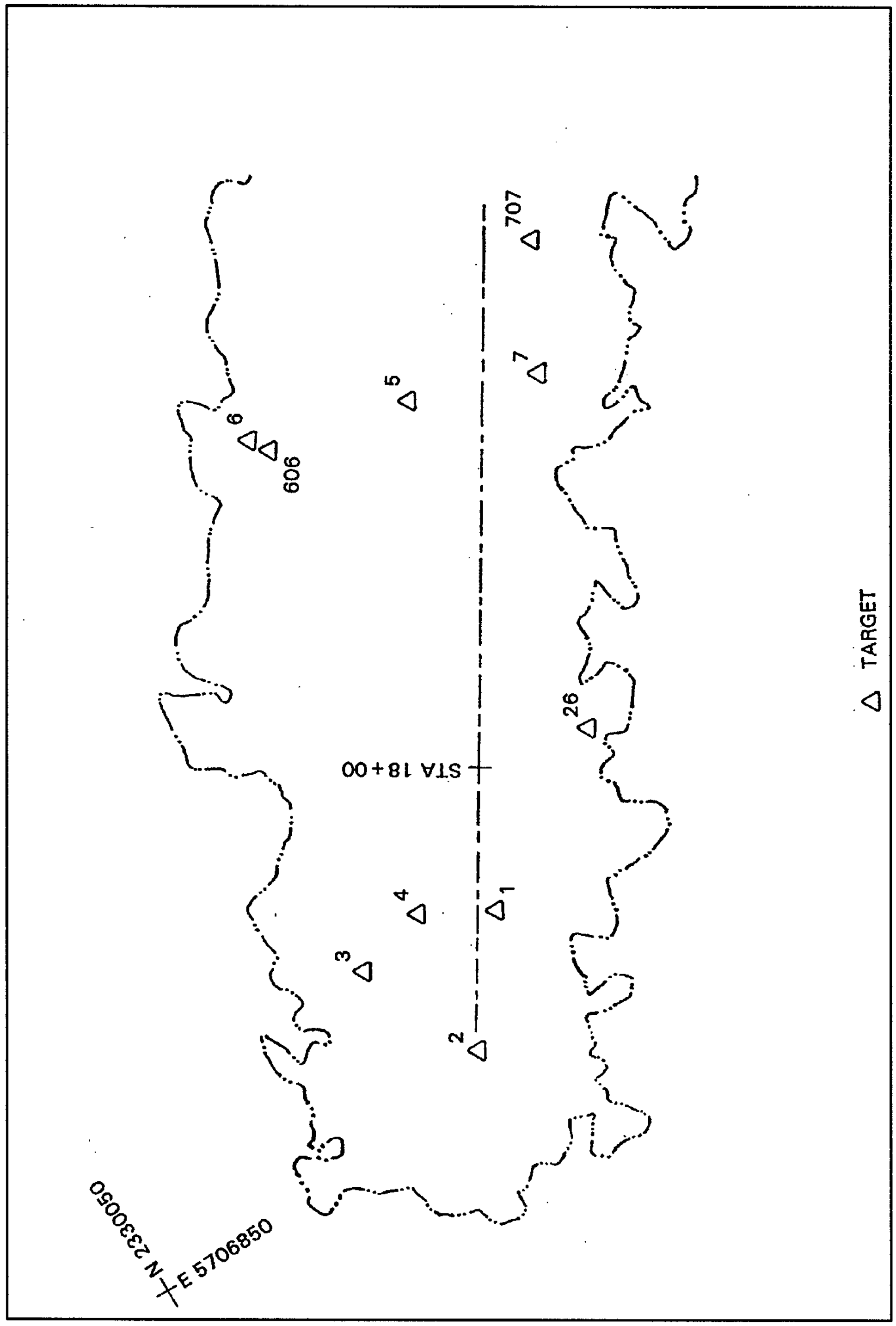

Figure D4. Locations of targets established on south breakwater, sta 17+10 – 18+50

Appendix E
Orthophotos, 2000

This appendix presents orthophotos obtained for the outer Morro Bay south breakwater as a result of the photogrammetric analysis conduced in July 2000. Orthophotos combine the image characteristics of a photograph with the geometric qualities of a map. The images have been rectified and are free from skewness and distortion. Precise horizontal measurements may be obtained from the orthophotos using an engineer scale. Station numbering is in an easterly to westerly direction. The scale of the maps is 2.54 cm = 6.1 m (1 in. = 20 ft).

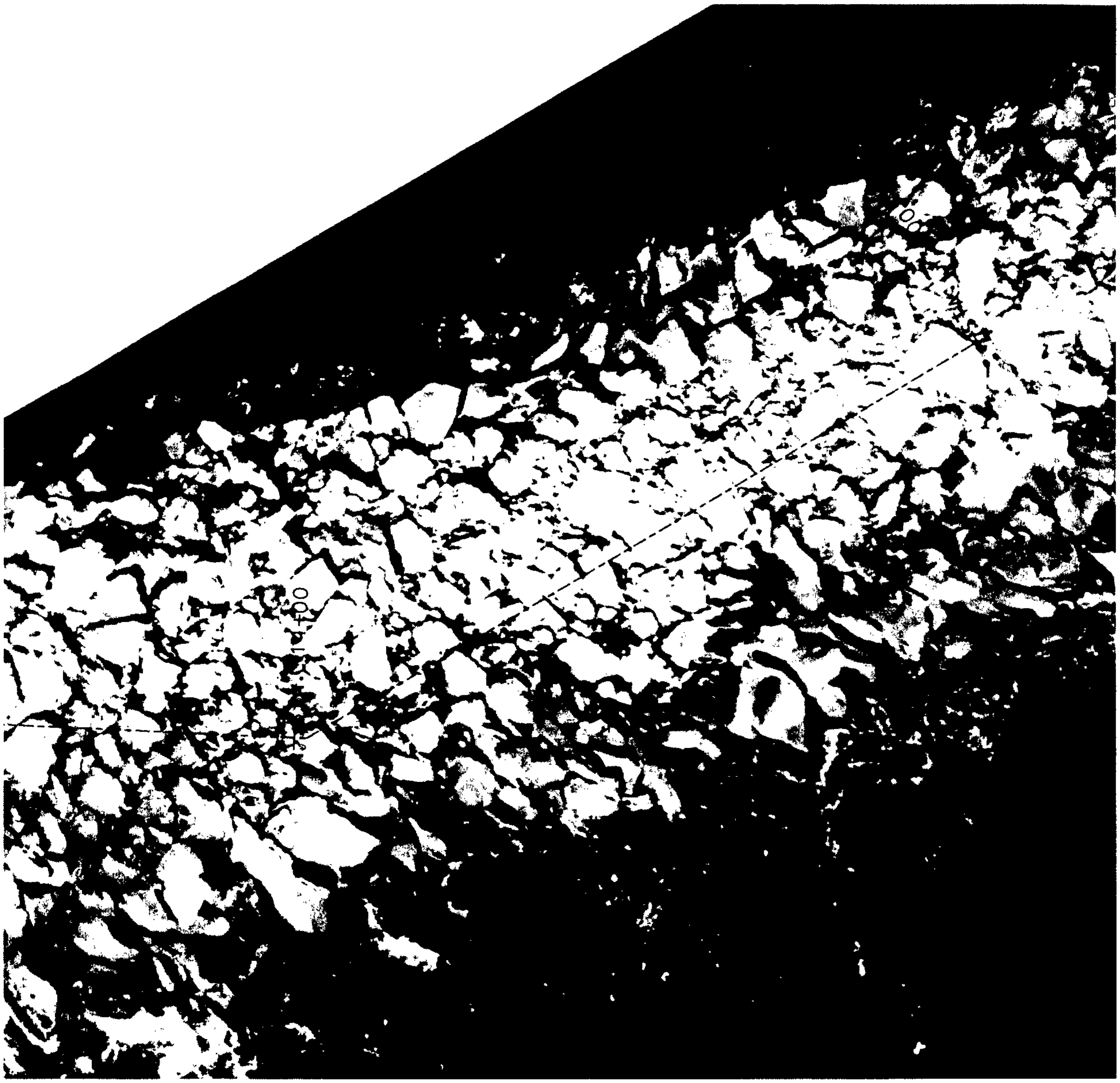

Figure E1. Orthophoto of outer south breakwater, July 2000, sta 13+00 – 14+40

Figure E2. Orthophoto of outer south breakwater, July 2000, sta 14+40 – 15+94

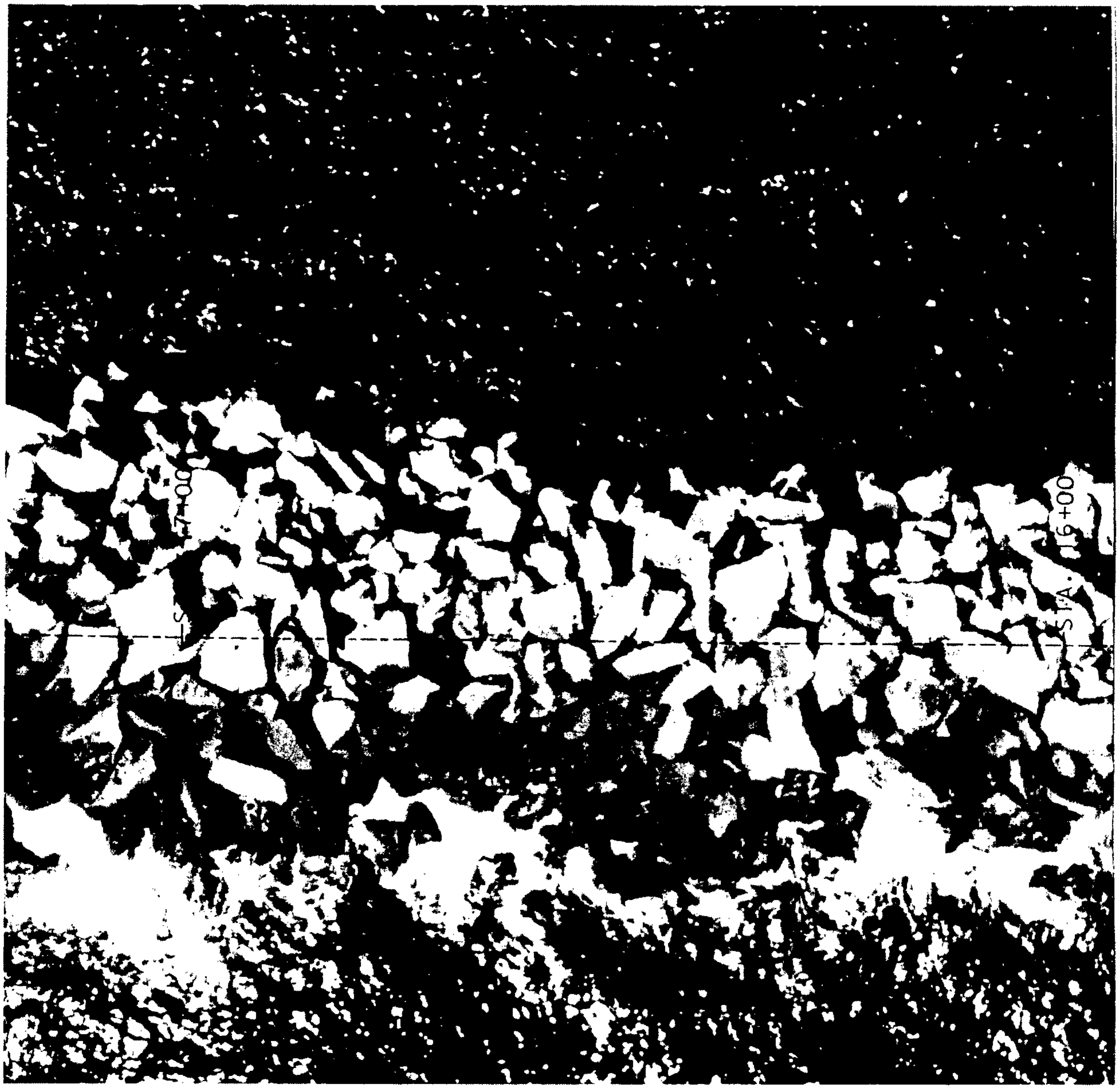

Figure E3. Orthophoto of outer south breakwater, July 2000, sta 15+94 – 17+20

Figure E4. Orthophoto of outer south breakwater, July 2000, sta 17+20 – 18+50

Appendix F
Breakwater Topography, 2000

This appendix presents contour maps of the outer Morro Bay south breakwater as a result of the photogrammetric analysis conducted in July 2000. Topography was developed using the digital terrain model (DTM) as stated in the main text of this report. The breakwater topography is shown on a 0.3-m (1.0-ft) contour interval. Elevations shown are in feet referred to mean lower low water (mllw) datum. To convert feet to meters, multiply by 0.3048. Station numbering on the contour maps is in an easterly to westerly direction. The scale of the maps is 2.54 cm = 6.1 m (1 in. = 20 ft).

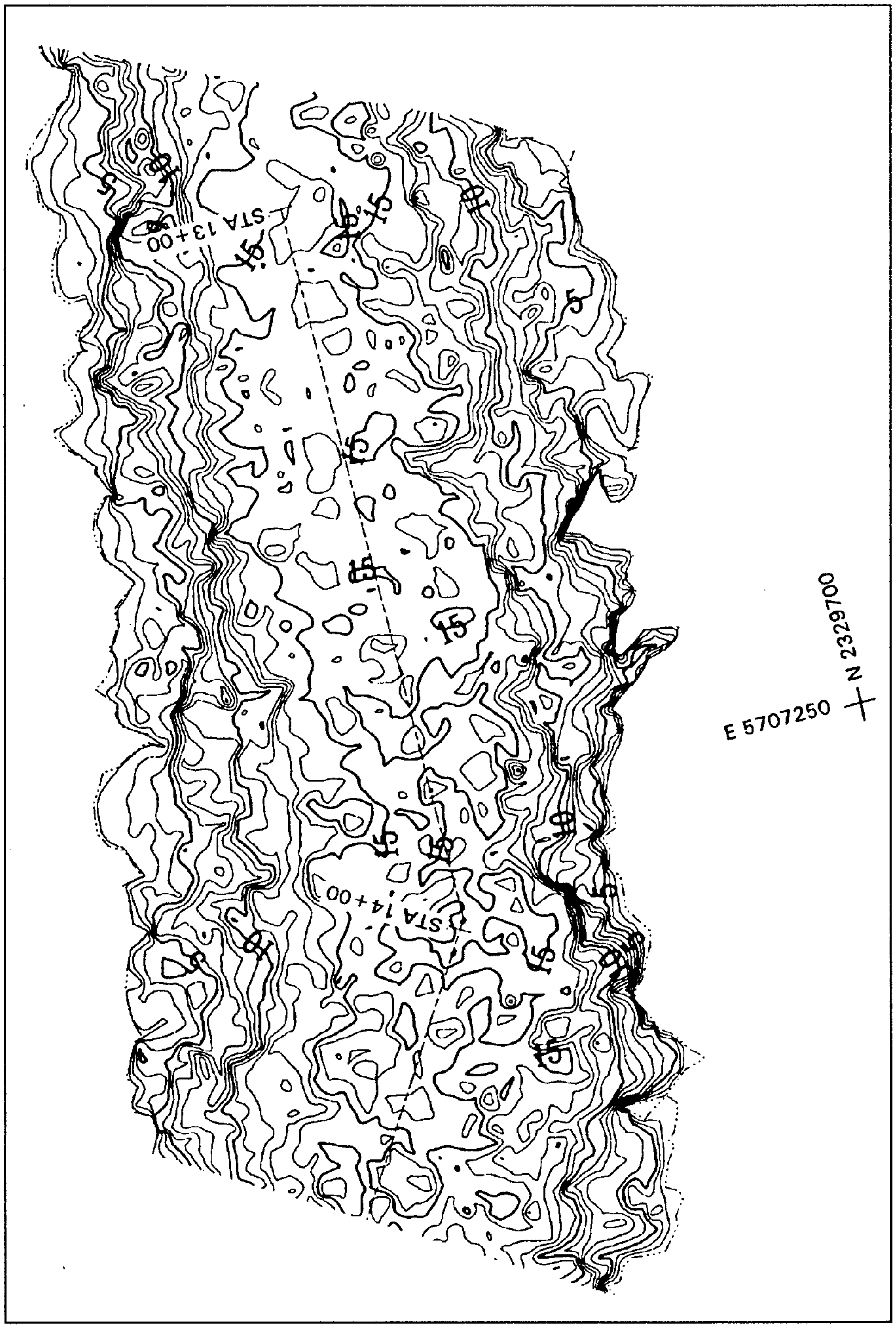

Figure F1. Topography of outer south breakwater, July 2000, sta 13+00 – 14+40

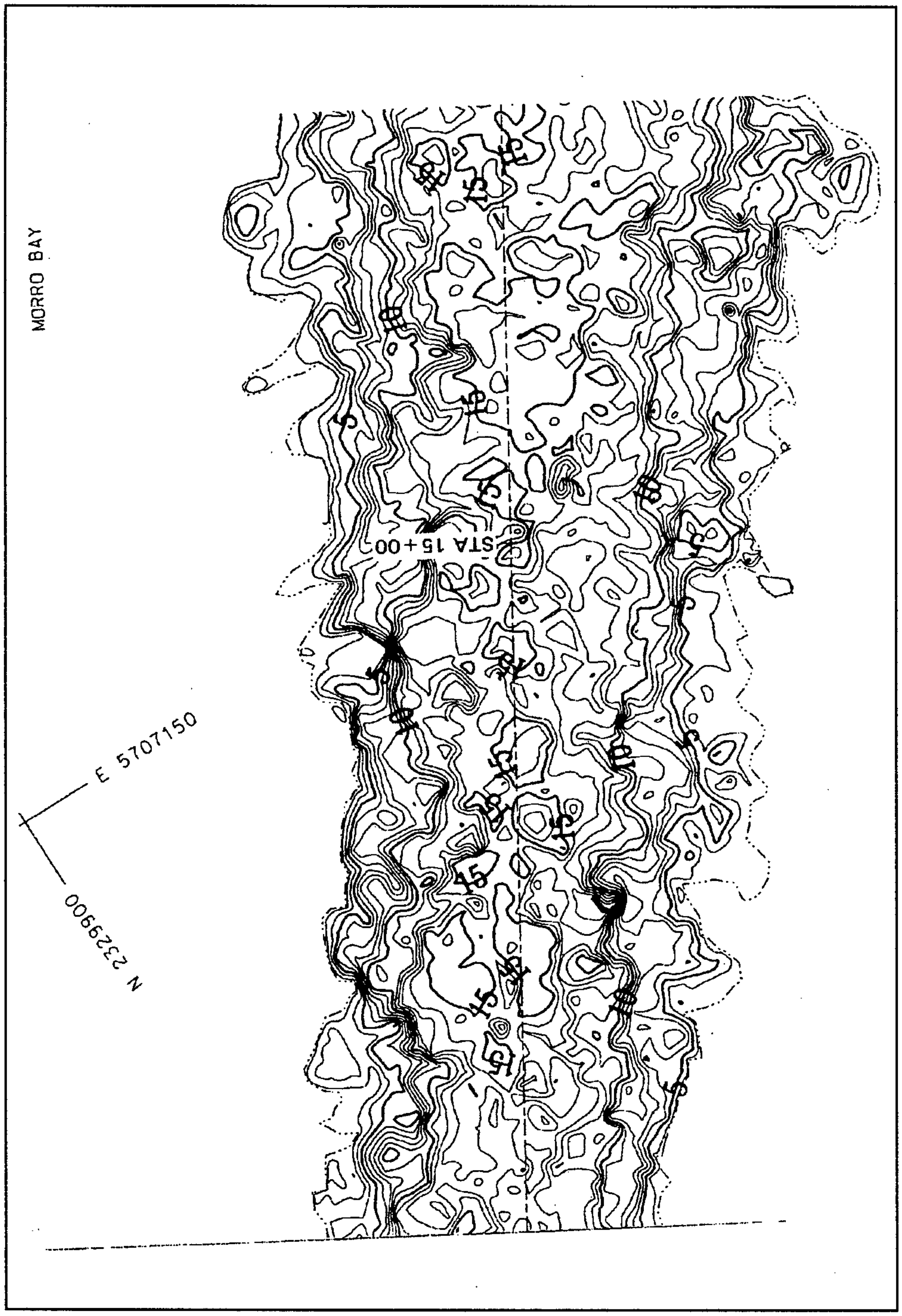

Figure F2. Topography of outer south breakwater, July 2000, sta 14+40 – 15+94

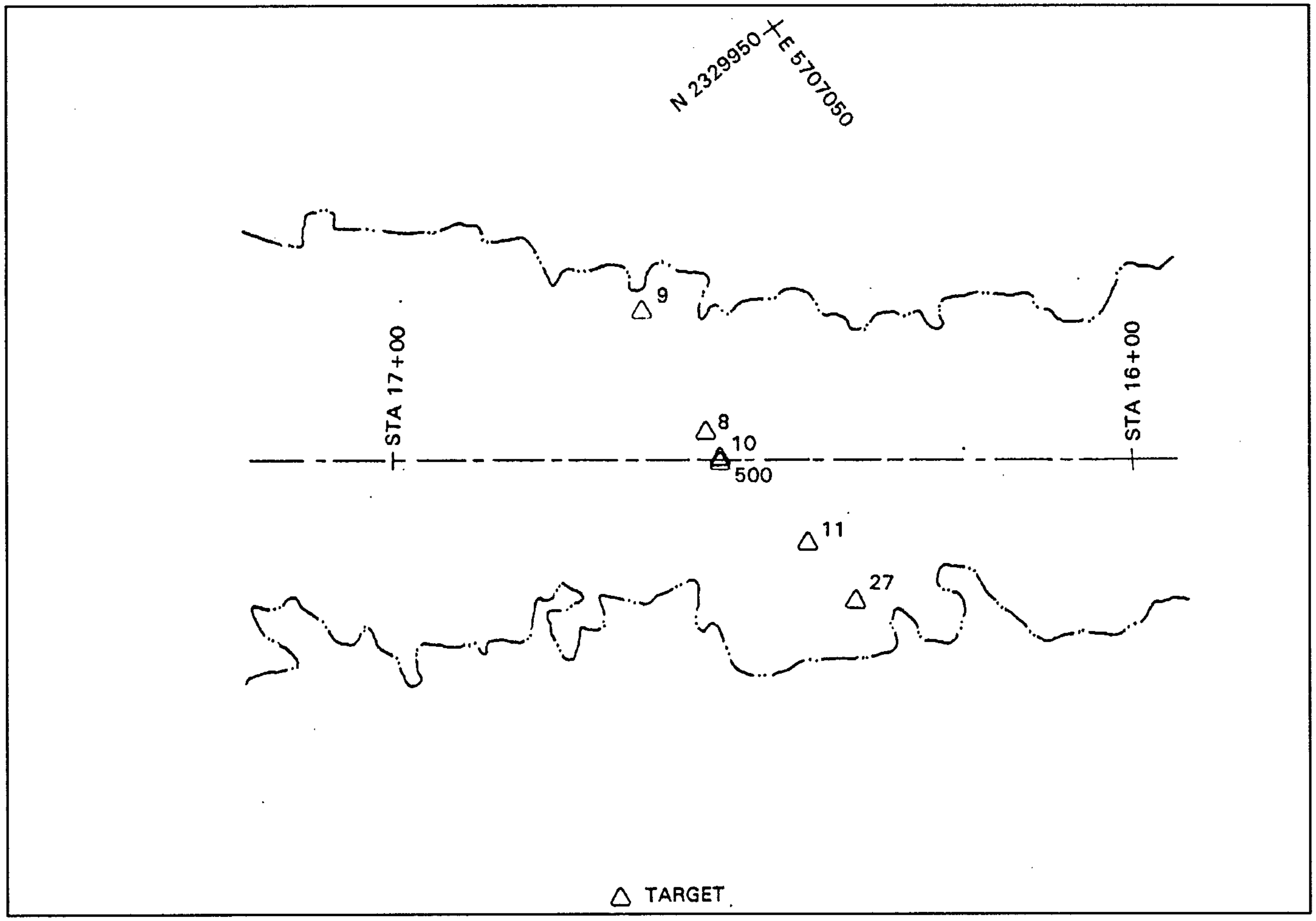

Figure F3. Topography of outer south breakwater, July 2000, sta 15+94 – 17+20

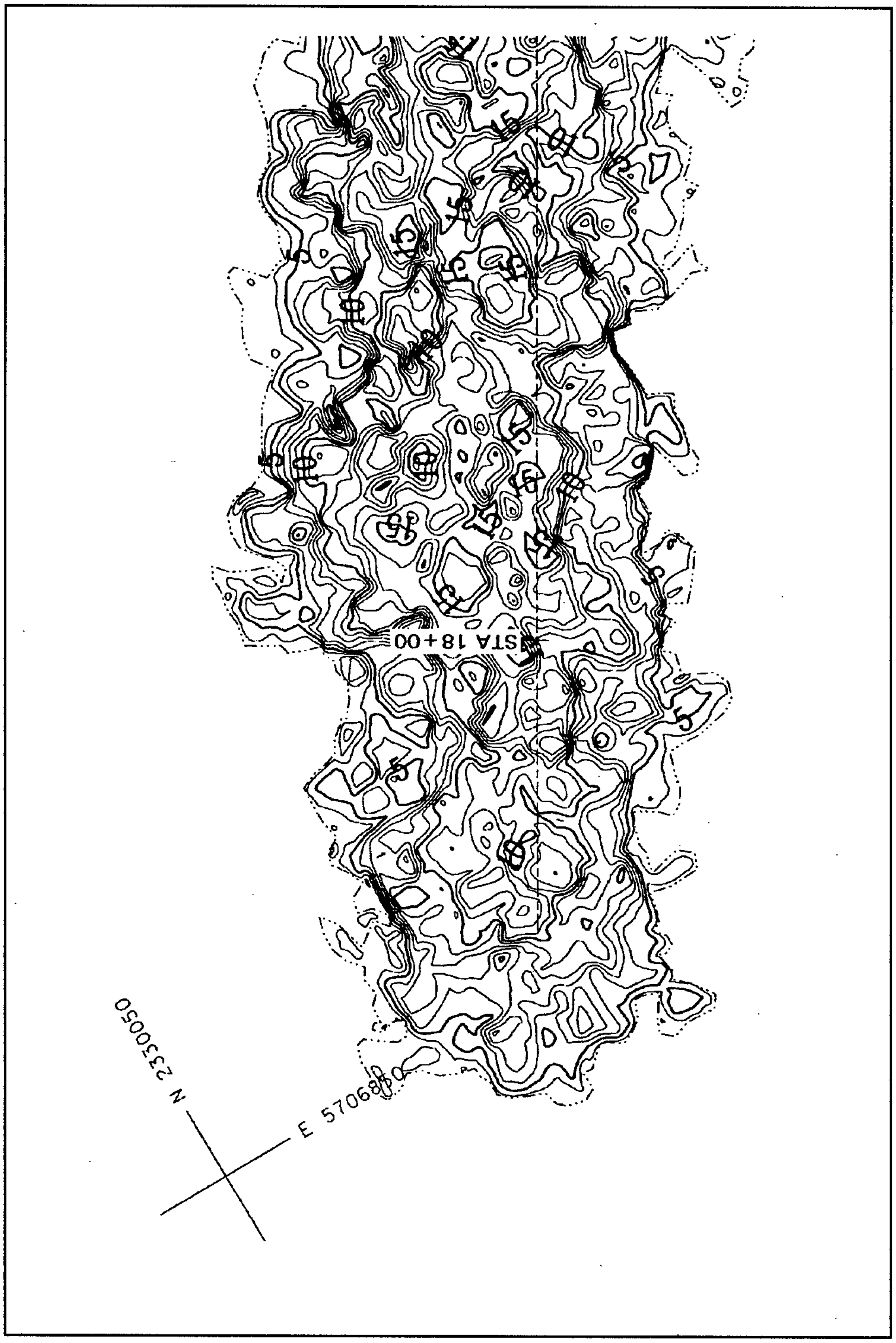

Figure F4. Topography of outer south breakwater, July 2000, sta 17+20 – 18+50

Appendix G
Locations of Individual Stone High Points

This appendix presents the locations of high points on 122 individual armor stones throughout the monitored structure. These points were selected from the photogrammetric stereomodel and elevations were compared for the 1998 and 2000 surveys. The scale of the maps is 2.54 cm = 6.1 m (1 in. = 20 ft).

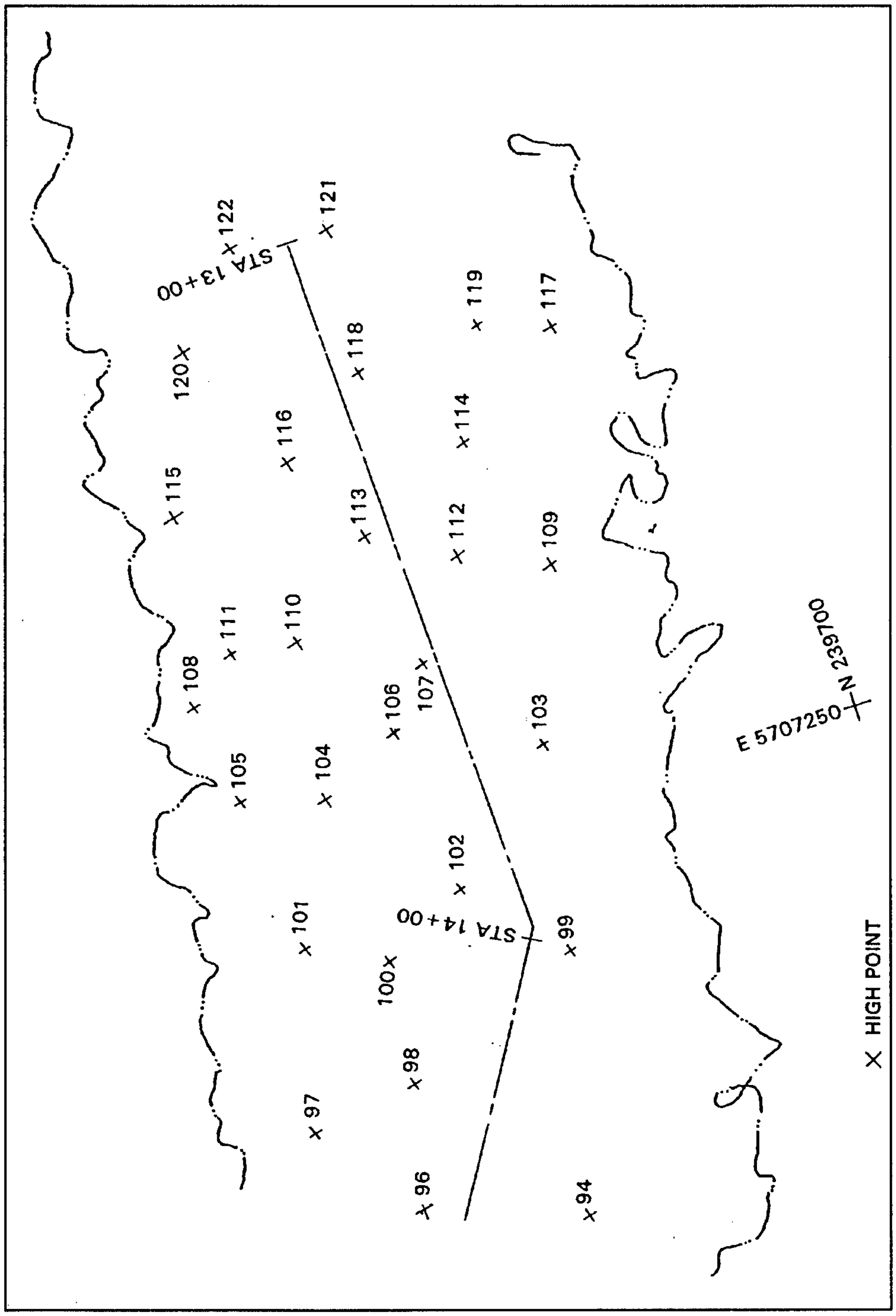

Figure G1. Locations of high points on south breakwater, sta 13+00 – 14+40

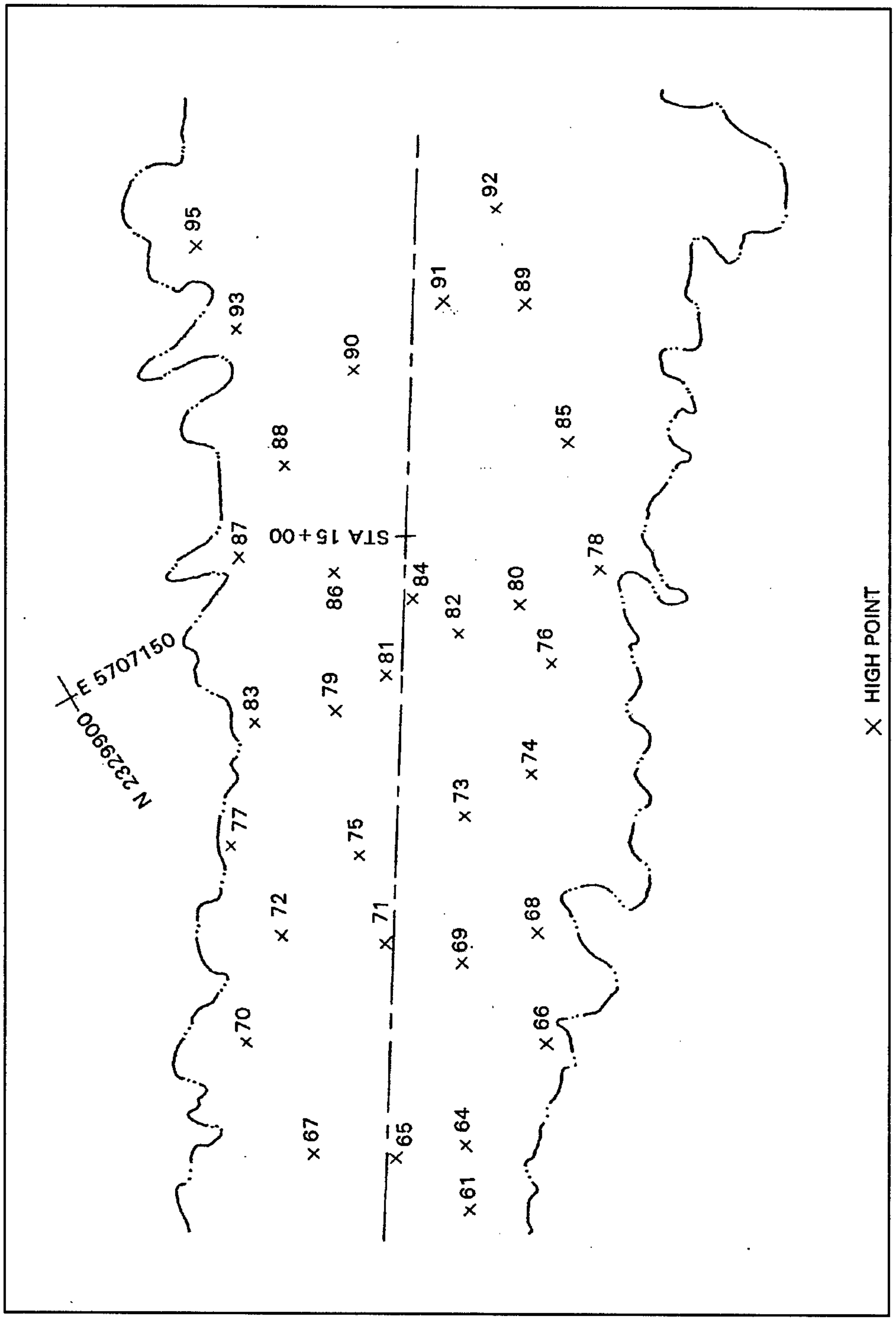

Figure G2. Locations of high points on south breakwater, sta 14+40 – 15+94

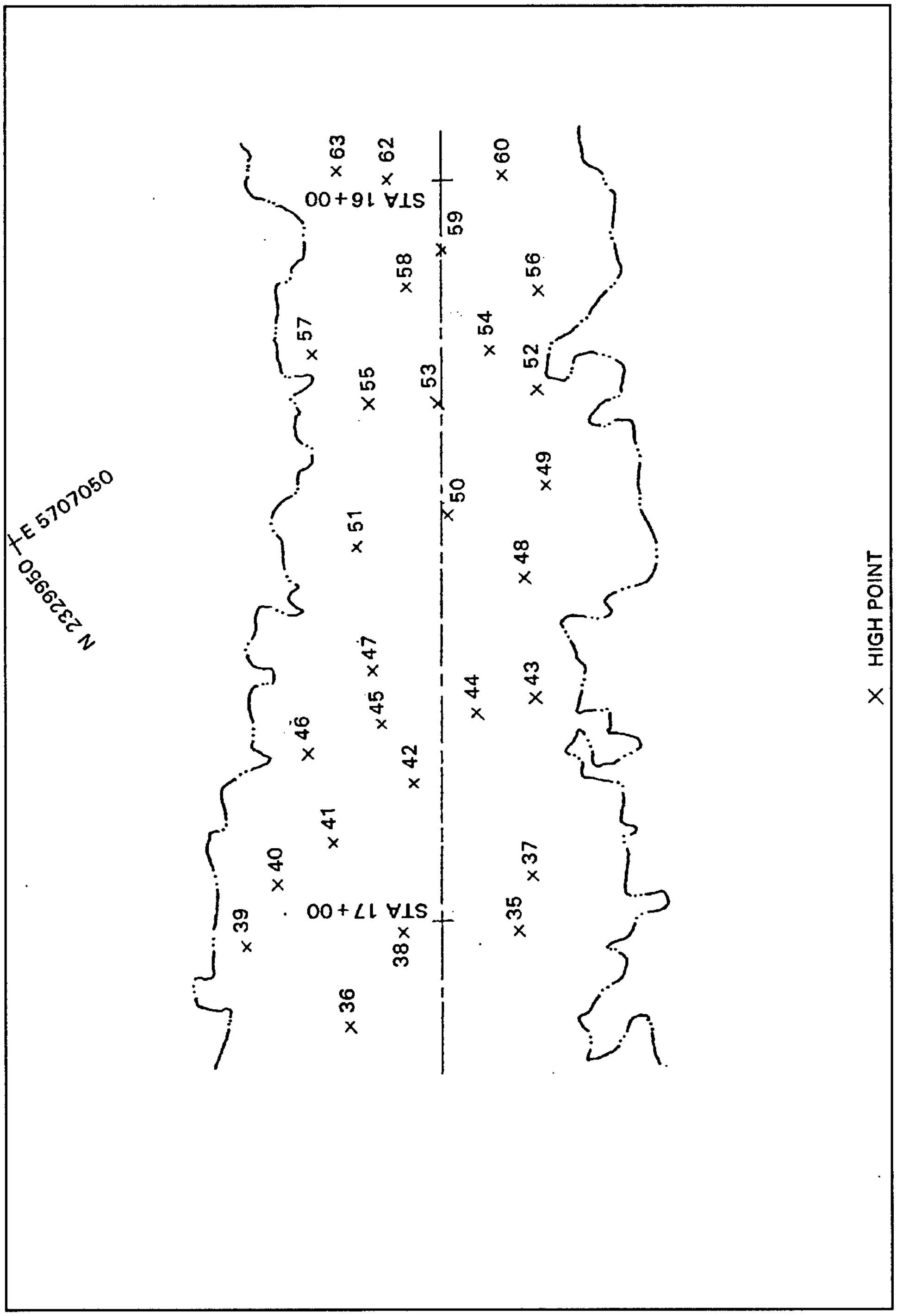

Figure G3. Locations of high points on south breakwater, sta 15+94 – 17+20

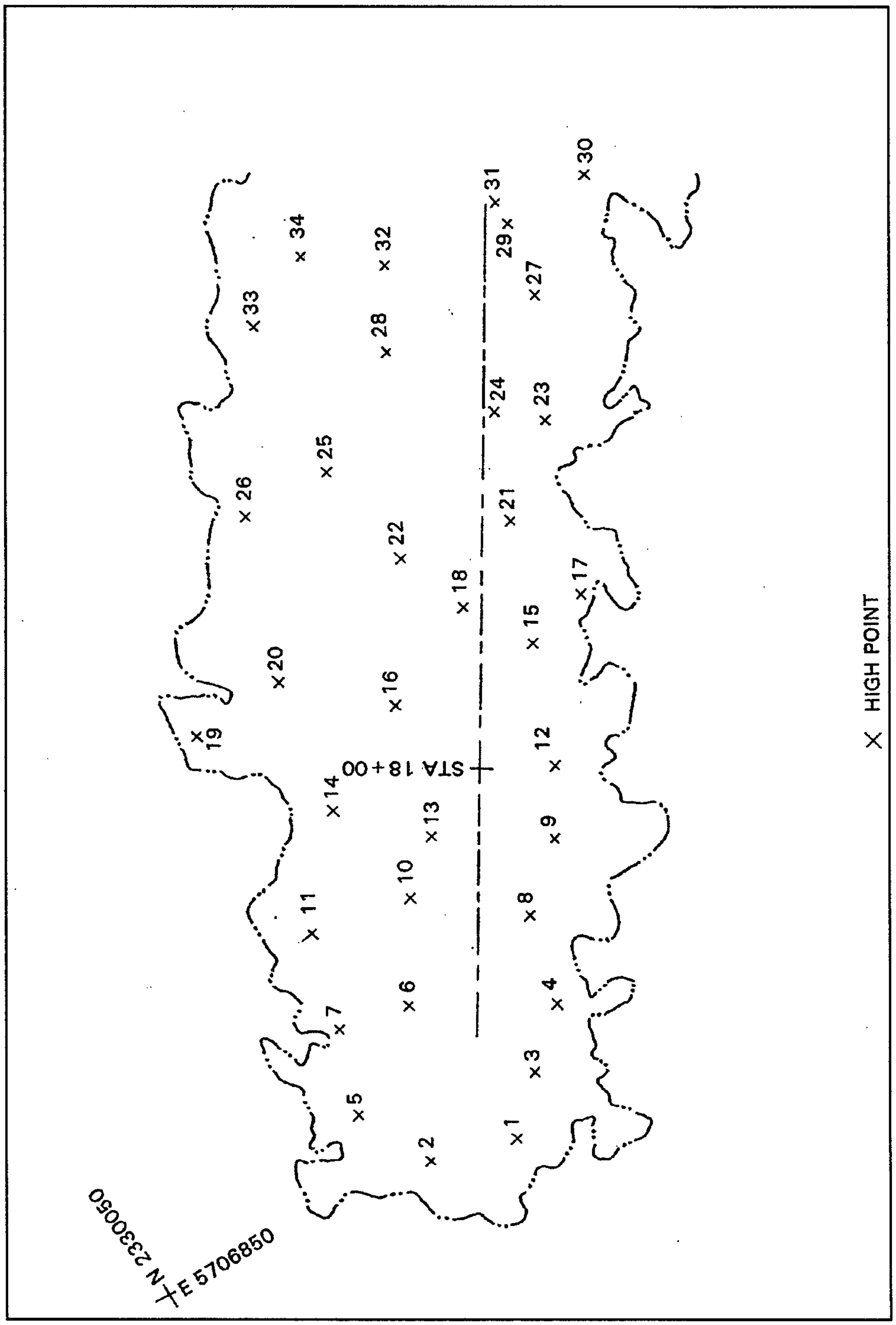

Figure G4. Locations of high points on south breakwater, sta 17+20 – 18+50

Appendix H Breakwater Cross Sections, 1998 and 2000

This appendix presents cross sections of the outer Morro Bay south breakwater for the 1998 and 2000 surveys. Cross sections were developed using the digital terrain model (DTM) grid as stated in the main text of this report. They were obtained at 15.2-m (50-ft) intervals along the outer 168-m (550-ft) length of the breakwater. Elevations shown are in feet referred to mean lower low water (mllw) datum. Distances from the baseline also are shown in feet. To convert feet to meters, multiply by 0.3048. Negative distances are measured relative to the sea side of the baseline and positive distances are measured relative to the harbor side of the baseline.

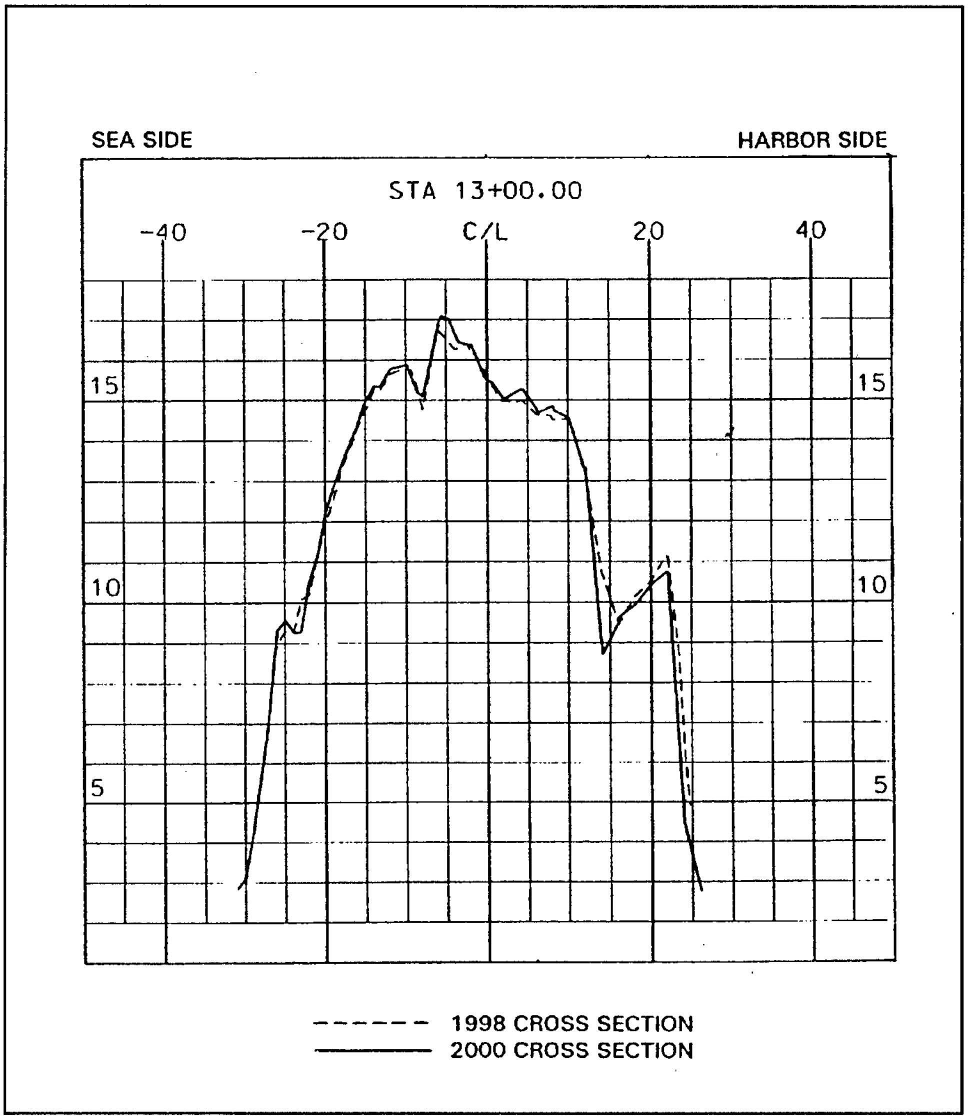

Figure H1. Cross sections of Morro Bay outer south breakwater, sta 13+00

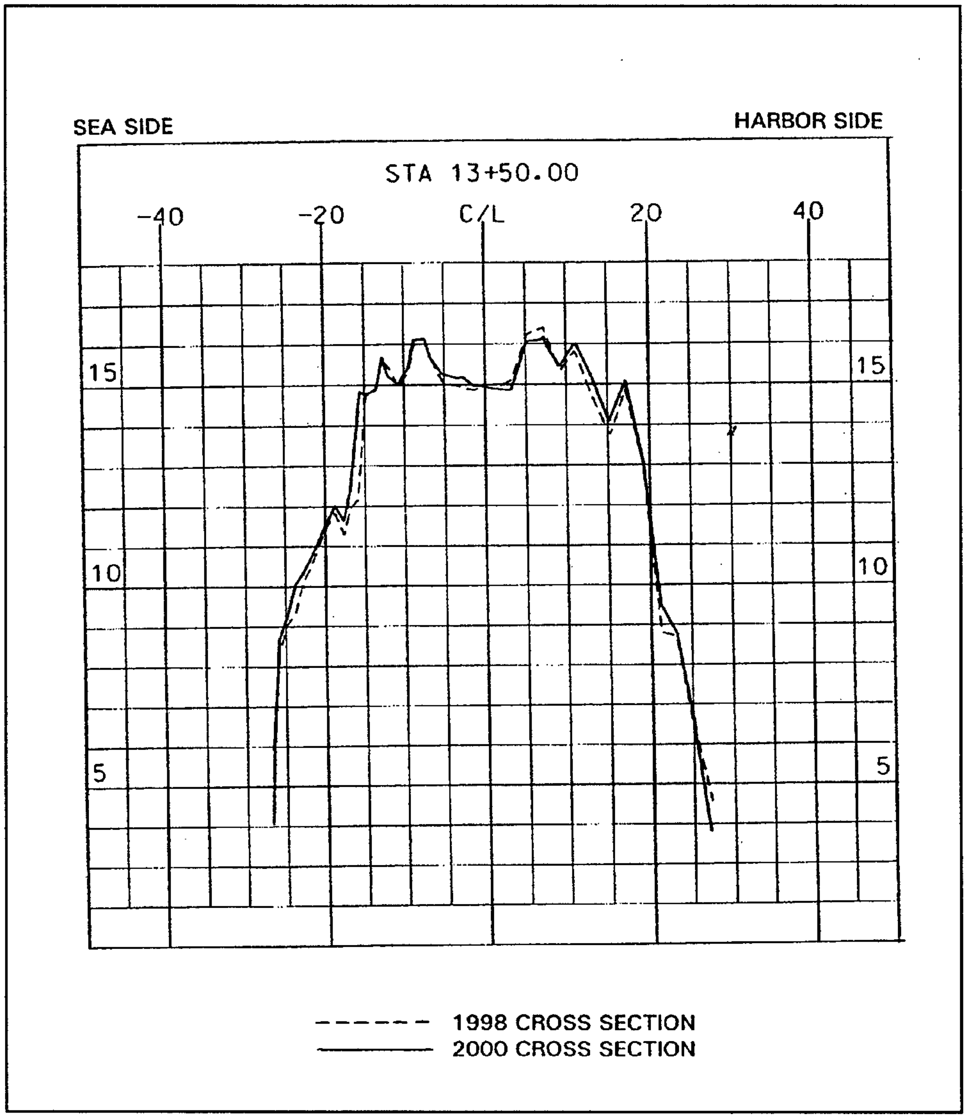

Figure H2. Cross sections of Morro Bay outer south breakwater, sta 13+50

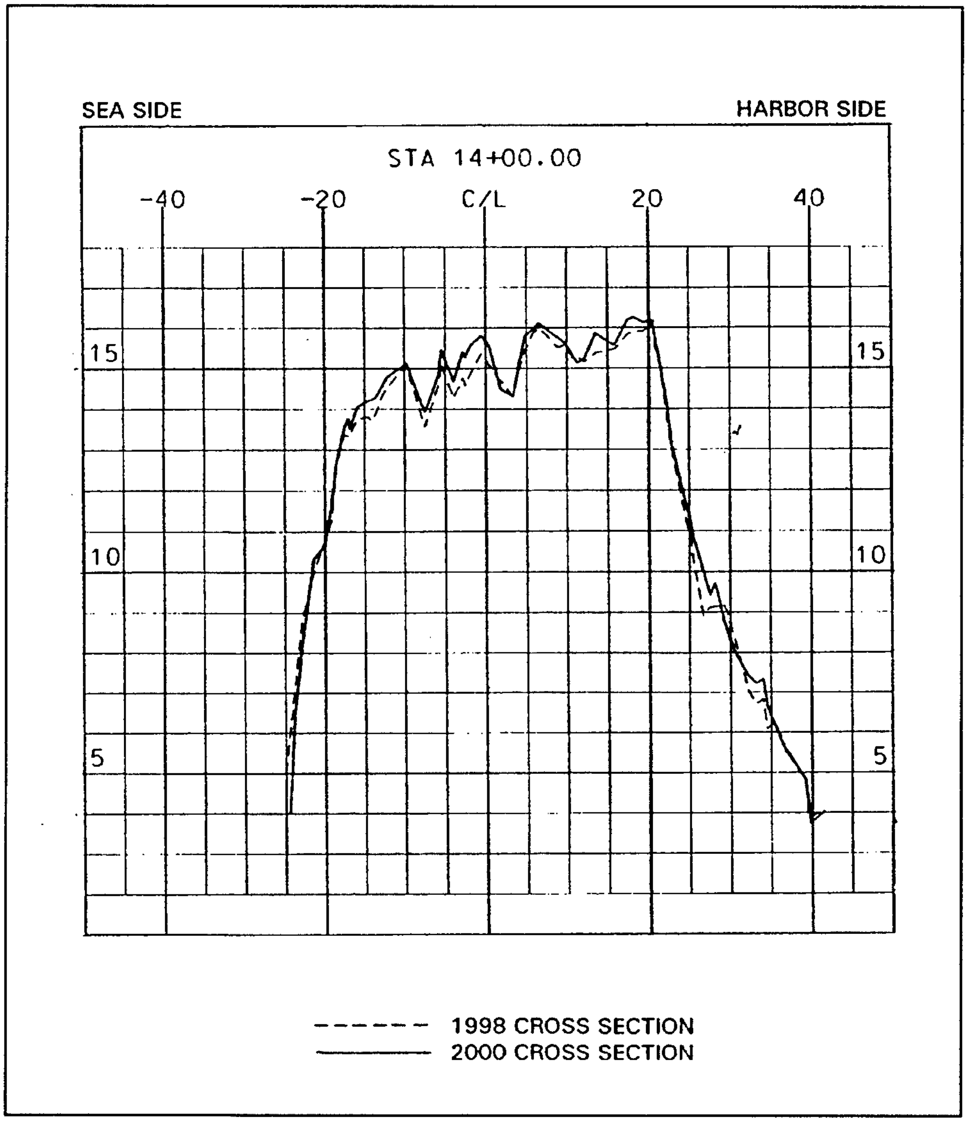

Figure H3. Cross sections of Morro Bay outer south breakwater, sta 14+00

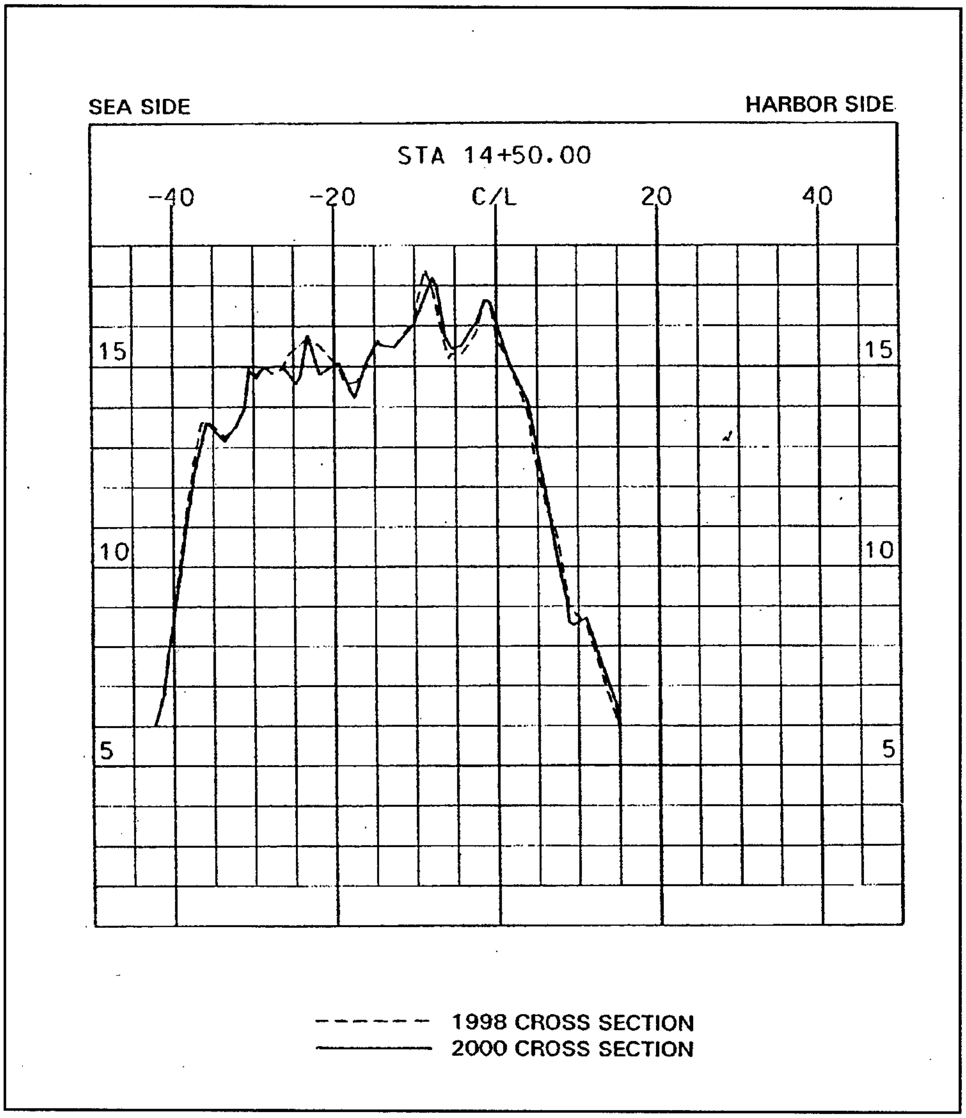

Figure H4. Cross sections of Morro Bay outer south breakwater, sta 14+50

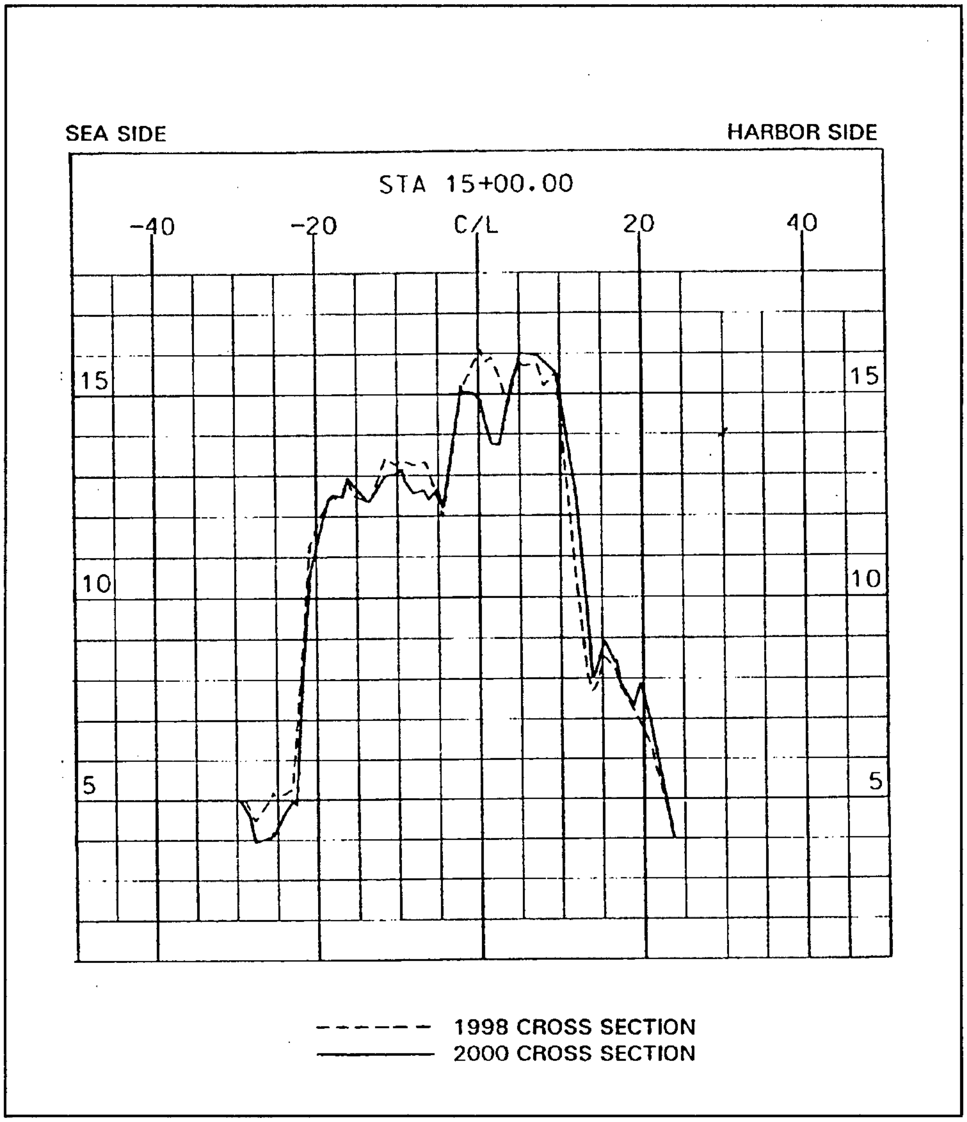

Figure H5. Cross sections of Morro Bay outer south breakwater, sta 15+00

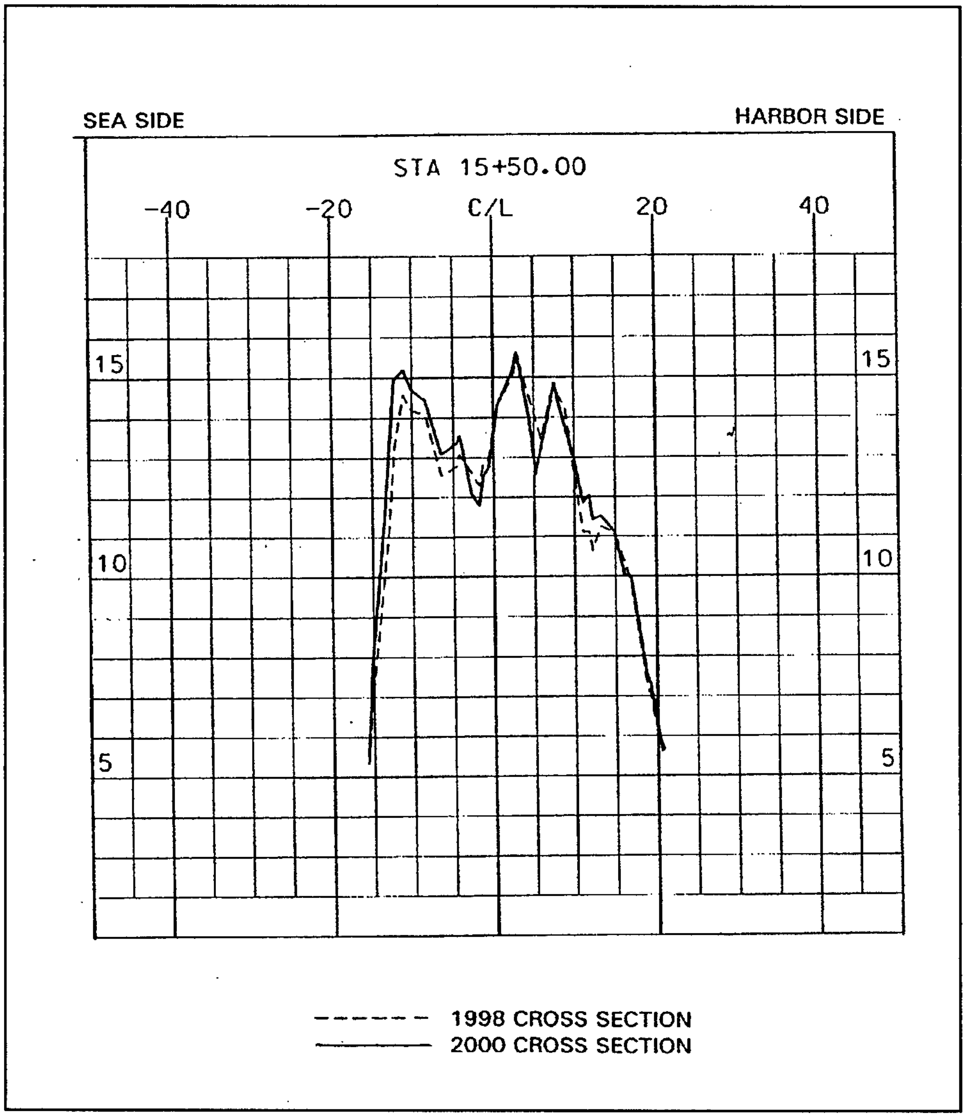

Figure H6. Cross sections of Morro Bay outer south breakwater, sta 15+50

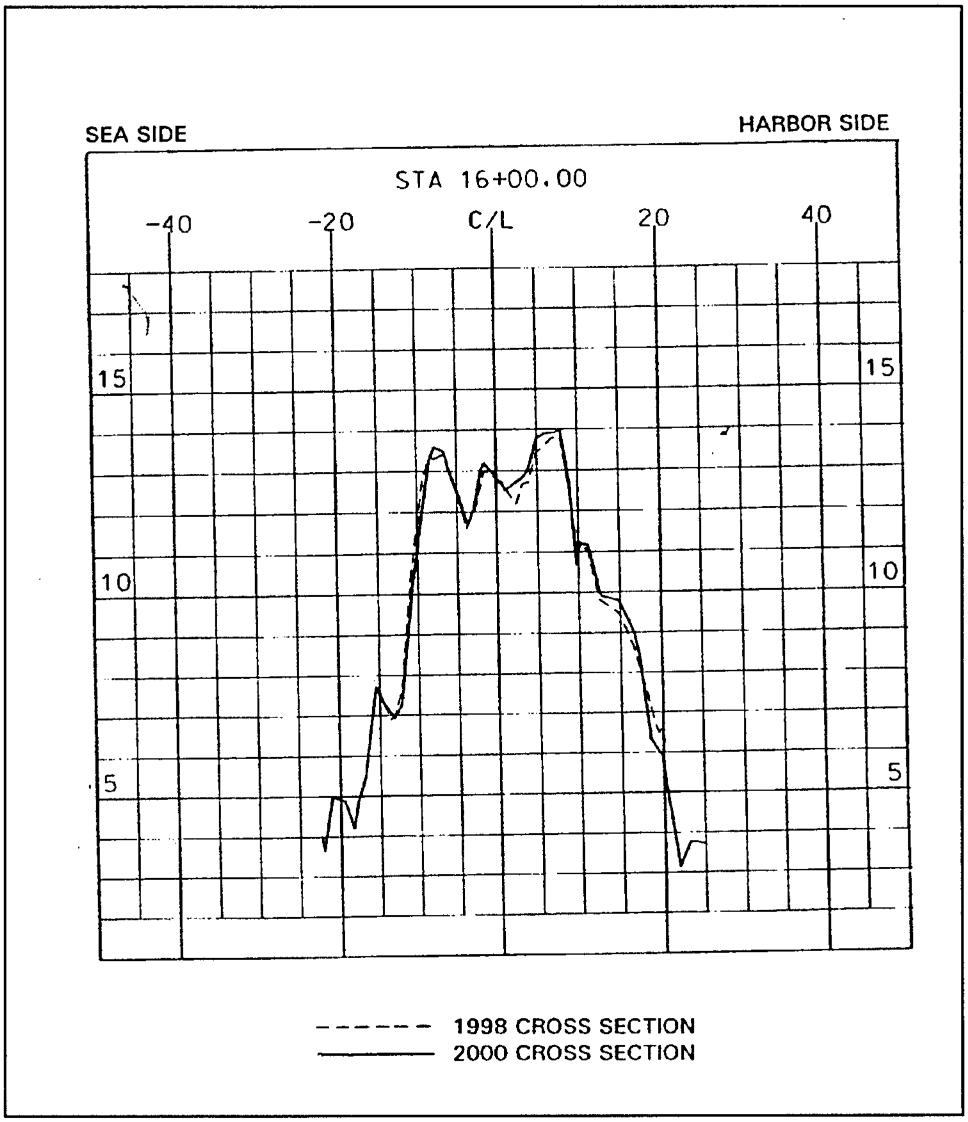

Figure H7. Cross sections of Morro Bay outer south breakwater, sta 16+00

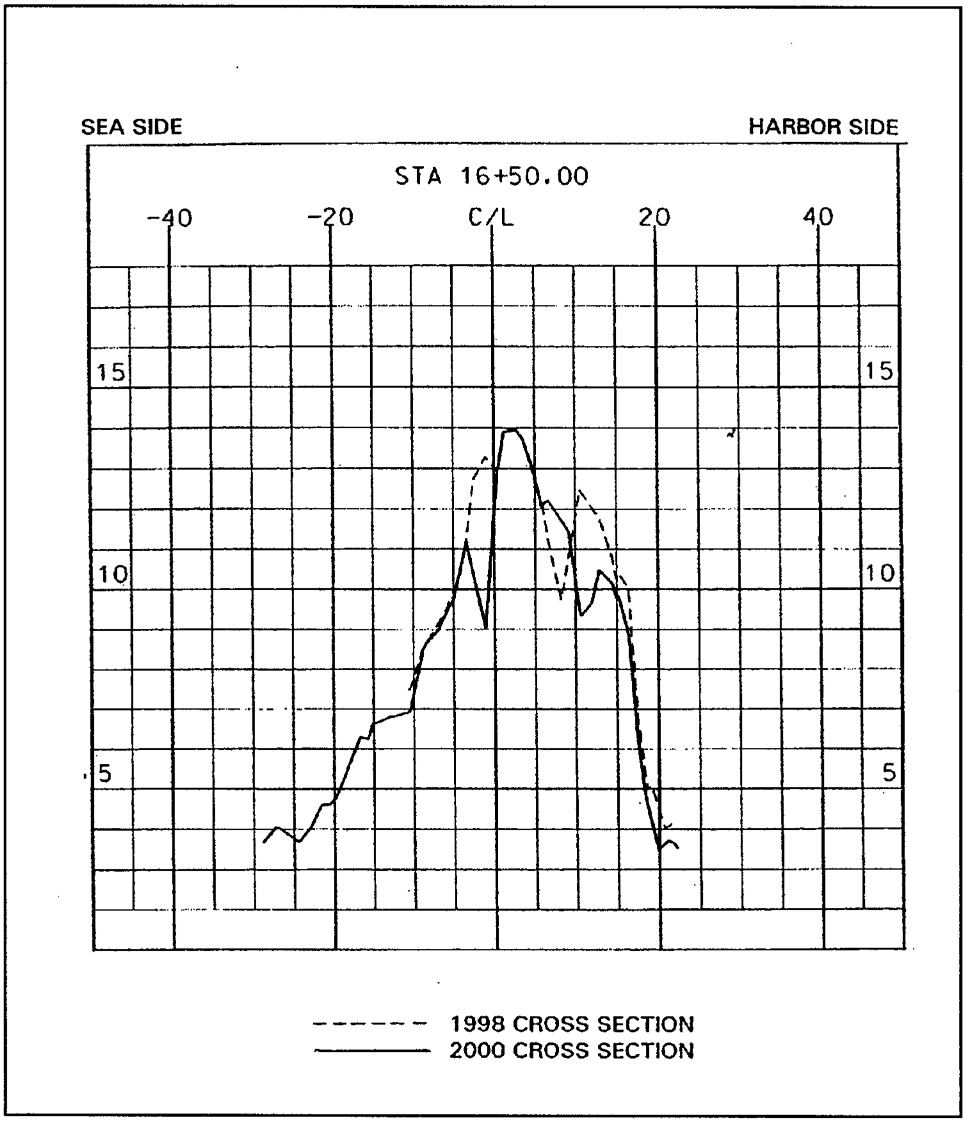

Figure H8. Cross sections of Morro Bay outer south breakwater, sta 16+50

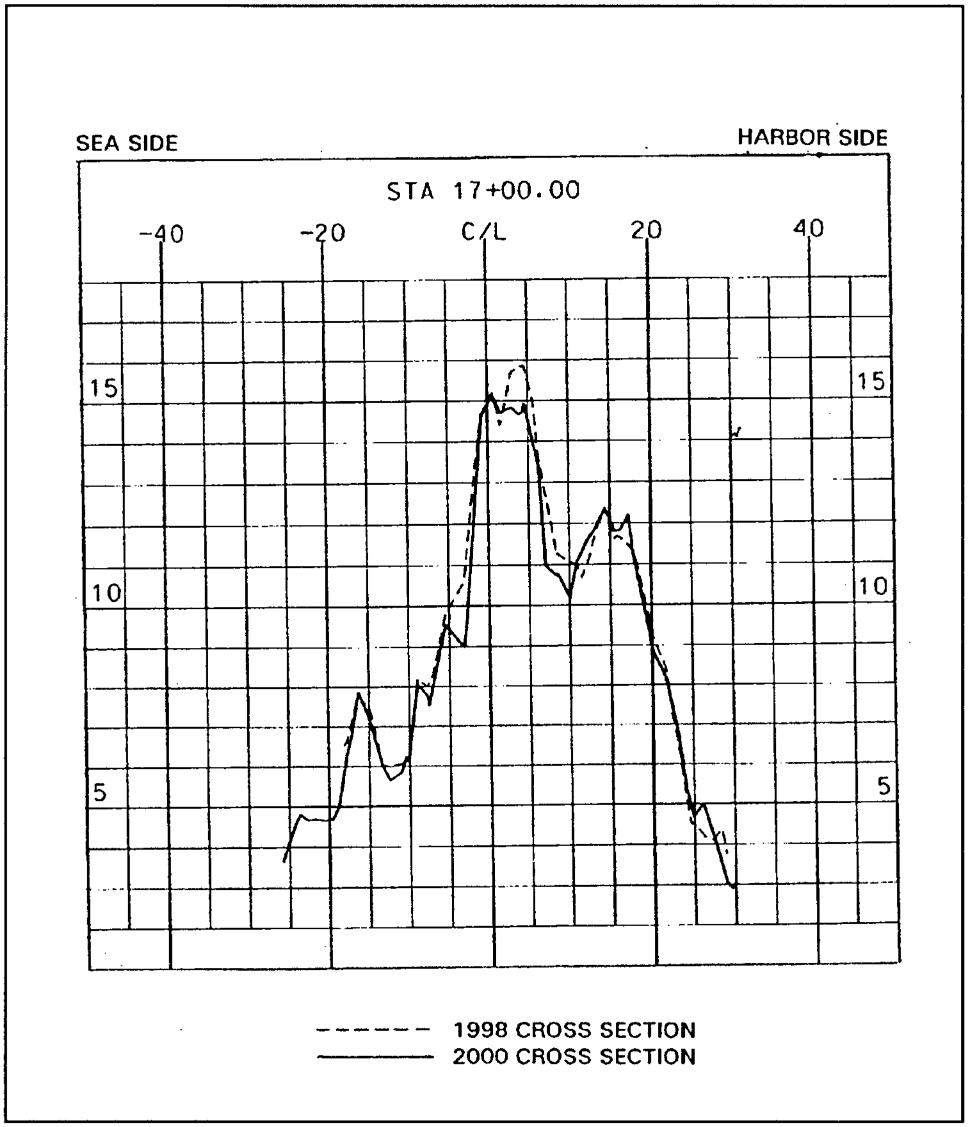

Figure H9. Cross sections of Morro Bay outer south breakwater, sta 17+00

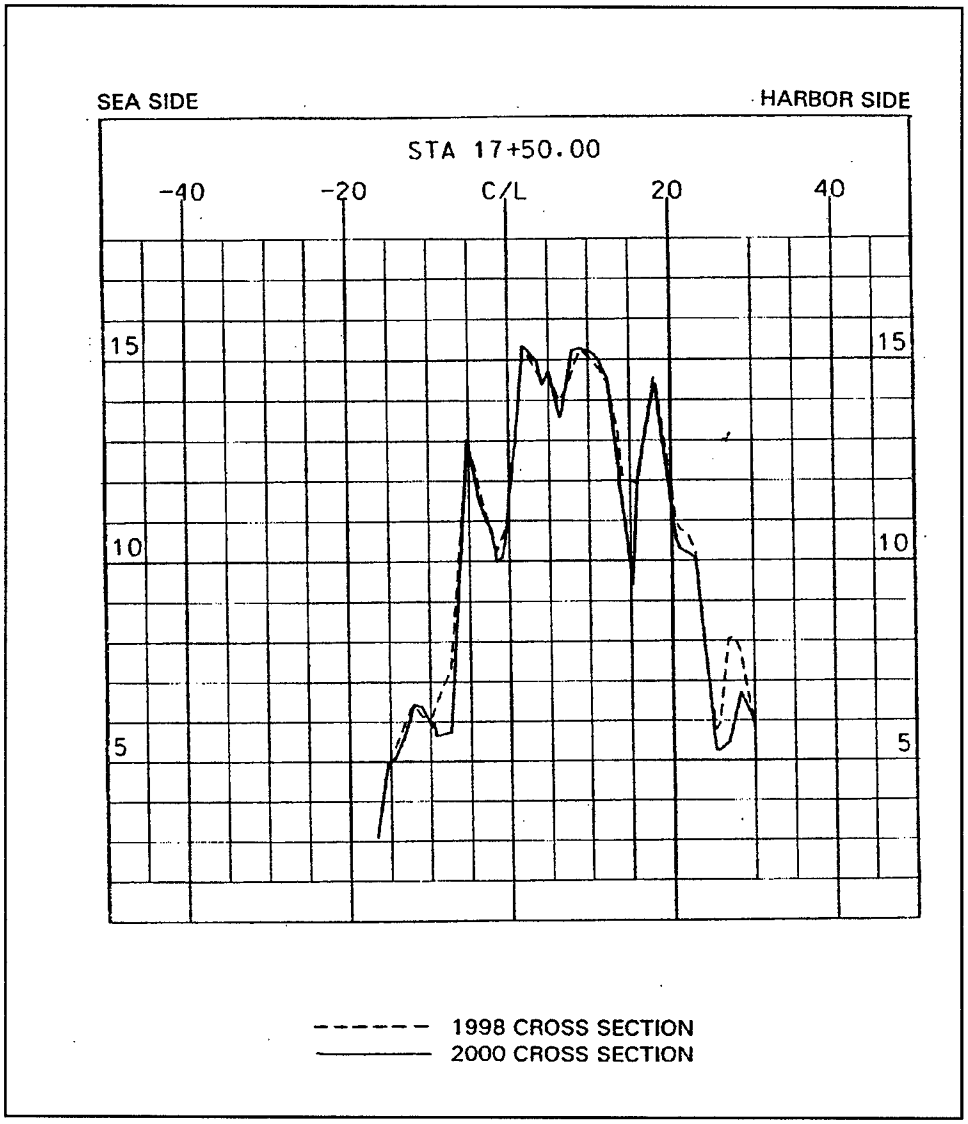

Figure H10. Cross sections of Morro Bay outer south breakwater, sta 17+50

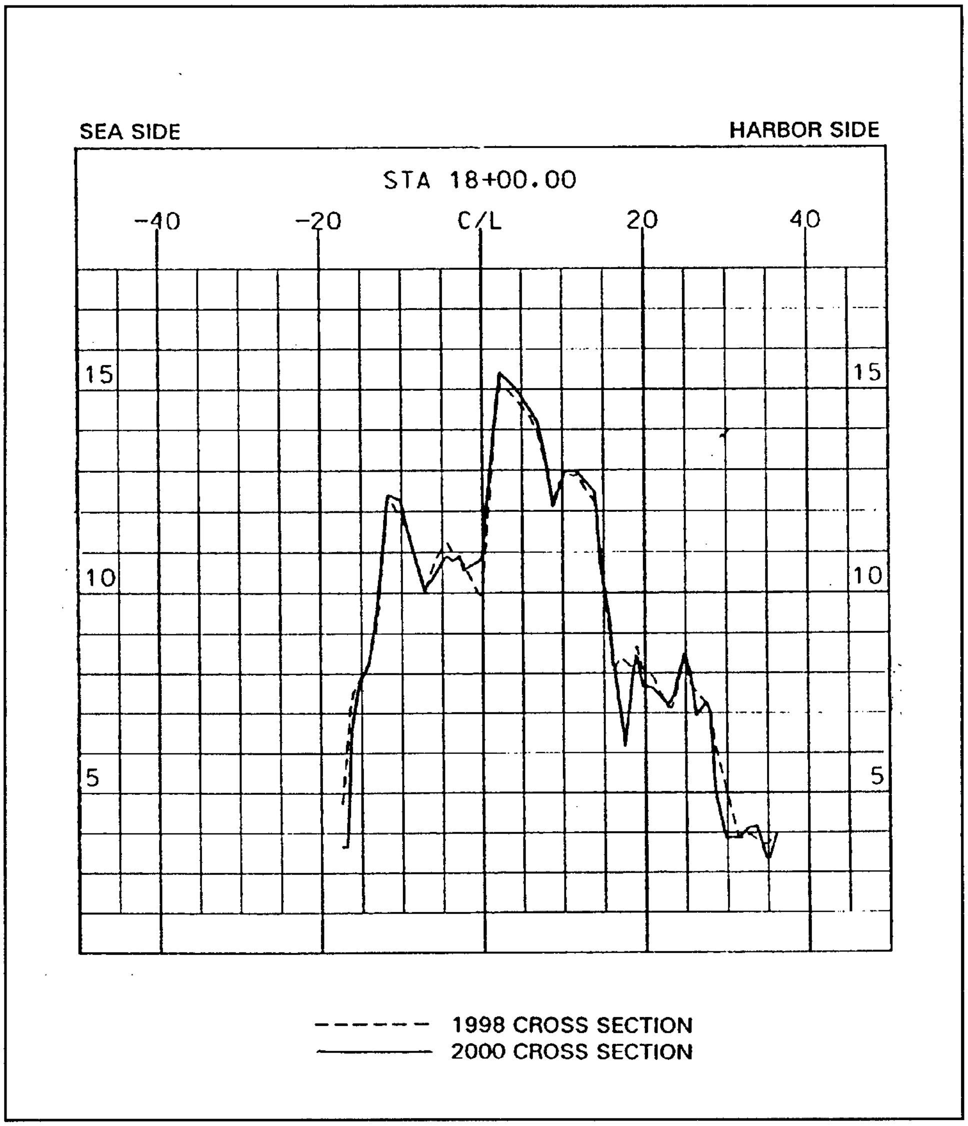

Figure H11. Cross sections of Morro Bay outer south breakwater, sta 18+00

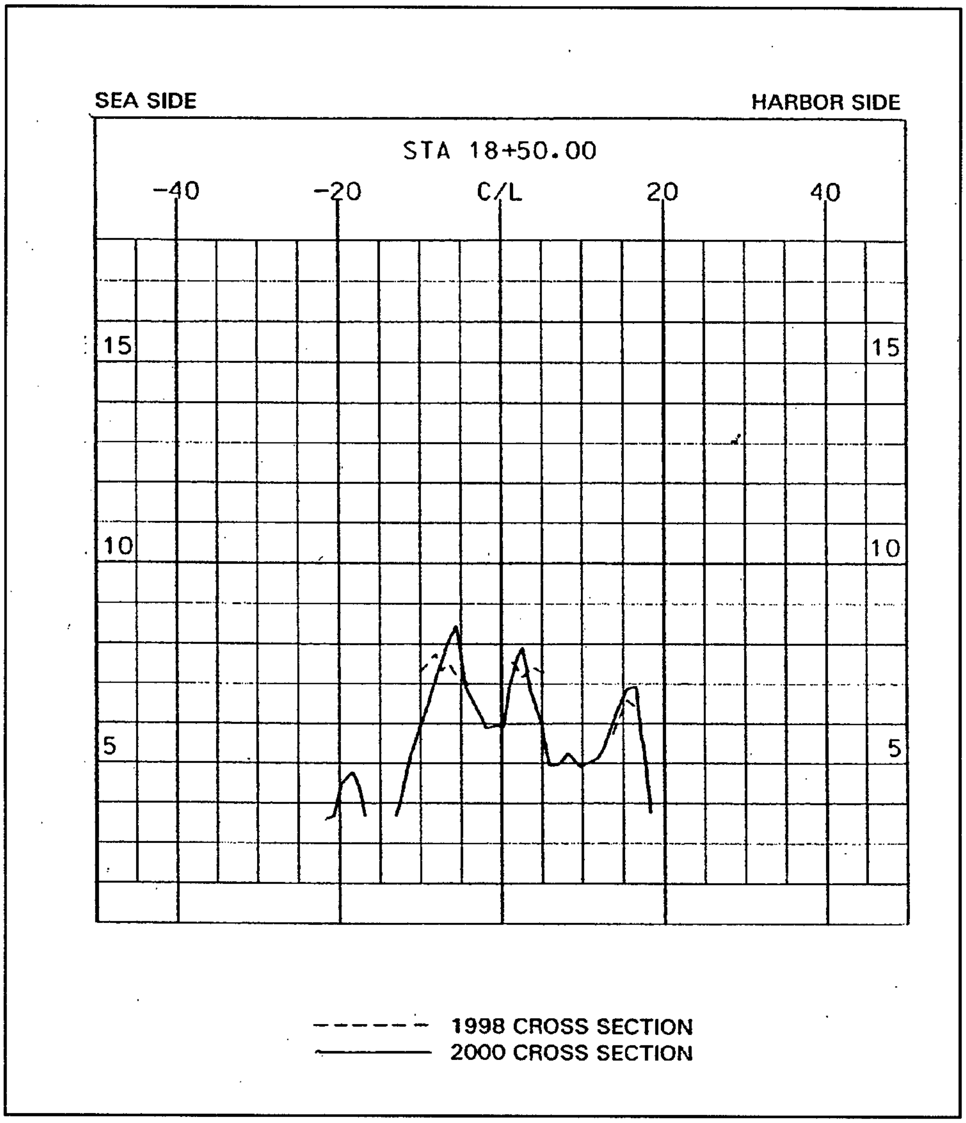

Figure H12. Cross sections of Morro Bay outer south breakwater, sta 18+50

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14. ABSTRACT

In 1997, Morro Bay Harbor, CA, was accepted for inclusion in the Monitoring Completed Navigation Projects Program. The objective of the monitoring effort at Morro Bay Harbor was to determine if modifications at the harbor entrance were performing as predicted by model studies used in the project design. Monitoring of the harbor entrance was conducted during the period January 1998 through August 2001. Elements of the monitoring program included the collection of wave data, tidal data, current data, and bathymetric data as well as a ground-based survey and photogrammetry of the south breakwater. It was concluded that improvements constructed at the Morro Bay Harbor entrance resulted in improved navigation conditions and had no negative impact on the existing structures. It was also determined that model investigations used in the design of the project accurately quantified wave conditions in the entrance and correctly defined sediment patterns and deposition areas. In addition, the study indicated that improvements can be effectively maintained with a 3-year dredging interval in the future.

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Orthophotos
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Sediment patterns
Sediment transport
Sedimentation rates
Shoaling
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Wave heights